AN INVESTIGATION INTO BOND STRENGTH IMPORTANCE IN FERRO-CEMENT

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The effect of variations in the strength of the wire-mortar interface bond on the properties of steel wire reinforced portland cement mortar is investigated in theory and experimentally. It is shown that the level of strength of this bond is of importance and that the bond on wires running across the axis of applied tensile stress is of vital importance. If the bond of these wires is impaired they are found to act as potent stress concentrators, giving rise to cracks that are positioned correctly for rapid propagation by the energy provided by the applied stress.

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by

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## INTRODUCTION

The recent widespread use of mesh-reinforced portland cement mortar for the construction of small marine craft has focussed the attention of the marine field on a material that has heretofore been of interest primarily to the civil engineer. The severe penalties associated with excess weight on the performance of marine vehicles and structures mitigate against the use of large factors of safety to insure adequate local strength. The catastrophic results of even minor local failure on a vehicle that depends on total integrity of structure for support on water demand knowledge of the exact method of failure to be expected in any material used in the construction of such a structure.

The advantages claimed by the proponents of mesh-reinforced mortar for such construction include increased flexibilities above those associated with normally-reinforced concrete and exceptional strengths. Before designing a structure that utilizes these qualities, any conscientious engineer should demand proof that these qualities do in fact exist. Past work by the author[15] and others[29] has indicated that these properties do not at present obtain in reproducible fashion.

Accordingly, there is a need to clarify the mechanism of failure in this material. This requires detailed examination of the material before, after and (ideally)

during failure. Only through such examination can the actual causes and preventatives of failure be understood and manipulated in the design of a material for a specific use with confidence.

During previous work, the author[15] observed supposedly similar samples that behaved in completely different manners. While some failed slowly, in a manner similar to the disruption of reinforced concrete, others exhibited a brittle nature, failing as a unit at much higher loads. It is the purpose of this thesis to investigate the effect and variation of the wire-mortar interface bond on the mode of failure of wire mesh reinforced portland cement mortar.

## REVIEW OF LITERATURE

### Review of Fracture Mechanics

Hydrated portland cement paste, the binding medium portion of portland cement concretes and mortars, exhibits brittle fracture behavior in tension. The nature of this sort of fracture received a large amount of attention during World War II when the stell used to build many of our ships showed similar brittle characteristics at low temperatures, with many sudden and catastrophic failures. The resulting investigations produced and refined the applications of the field of fracture mechanics to the design of full-scale structures of steel. The application of the discipline to concrete and mortars waited until 1961, but it is now fairly well conceded that this form of analysis is applicable to these materials [17] [21].

The concept basic to fracture mechanics is that of a balance between the energy required to expand a crack over a unit area and that energy released by the relaxation of elastic strain in the structure as a result of the crack motion. Griffith [1] [2] first postulated this energy balance for ideally brittle materials in the early 1920's. The rather simple formula relating these two rates of energy release at the onset of unstable crack propagation in the case of a degenerate elliptic hole in a thin brittle plate under plane tensile stress is:

$$(1) \qquad 2T = \frac{\pi c \sigma^2}{E}$$

where $T \equiv$ specific surface energy of the newly-formed fracture surface

$c \equiv$ one half the length of the elliptic crack

$\sigma \equiv$ the nominal tensile stress on the plate (force acting at a distance)

$E \equiv$ Young's modulus of elasticity

The equality of these two terms is a necessary condition for unstable crack propagation in an ideally brittle material. It implies that if energy is released at a greater rate than it is consumed during crack movement the crack may grow without further input of work to the structure from outside.

Irwin[3] [4] extended this theory to include the energy-absorbing action of plastic working of the material ahead of the advancing crack tip in materials that are not ideally brittle. He took account of the rate of work absorption resulting from plastic straining of the material ahead of the advancing crack. He added it to the rate of energy absorbed in the creation of the cleavage fracture surface and developed the following version of the criterion for unstable crack propagation:

$$(2) \qquad \frac{\partial W}{\partial c} + 2T = \frac{\pi c \sigma^2}{E}$$

where $\frac{\partial W}{\partial c} \equiv$ the rate of work absorption (with respect to crack dimension) associated with plastic straining ahead of the crack.

This rate of energy absorption by plastic working is on the order of 1000 times the specific surface energy of ductile materials such as copper.

Irwin further defined two new terms to facilitate manipulation of the resulting expanded equation:

$G \equiv$ strain energy release rate

$G_c \equiv$ that value of G existing at the onset of unstable crack propagation

The term G thus represents the right-hand side of equation (1) and $G_c$ is the left-hand side as modified to include the effects of plastic working.

For a given material that can be considered homogeneous on a macroscopic level, $G_c$ is a material property for a given type of loading while G is a function of loading level and geometry of the crack and its surroundings. The requirement for unstable crack propagation can now be written:

(3) $G = G_c \Rightarrow$ Unstable crack propagation

Although only the simplified case of plane stress has been considered here, other stress conditions are matters of geometry only and result in the inclusion of geometric variables and Poisson's ratio only. The important points to note are:

a. G is proportional to the square of the nominal stress

b. G is proportional to the dimensions of the crack

c. $G_c$ can be altered, for a given type of loading, only by a basic modification of the material itself, since it is a function of the specific fracture surface energy and the plastic working of the material ahead of the crack

Irwin[25] further defined a related term that allowed line combination of stress fields to determine their effects on crack propagation. He called this value "stress intensity factor K" and defined it as:

(4) $K = \sqrt{GE}$ for the case of plane stress

(5) $K = \sqrt{\frac{GE}{(1-\nu^2)}}$ for plane strain

where $\nu \equiv$ Poisson's ratio

It can be shown [25] that this stress intensity factor can be linearly superimposed to yield the resultant stress intensity factor at the crack tip.

(6) $K_{resultant} = K_1 + K_2 + K_3 + \cdots + K_n$

where $K_n$ is the $n^{th}$ stress intensity factor and addition is algebraic

The value of K existing at the onset of unstable crack propagation is designated $K_c$ and is analagous to $G_c$. The criterion for unstable crack propagation can thus be written as:

(7) $K = K_c \Rightarrow$ unstable crack propagation

Although these concepts spring from an analysis of material from the standpoint of linear elastic mechanics, and hence are applicable only to the specialized case of completely homogeneous materials with linear behavior in the elastic region, they have been successfully applied to the case of portland cement concretes and mortars.

The first such application was by Kaplan [17] in 1961, in which he worked with notched concrete beams and calculated the values for the change of compliance with progressing crack length during fracture. These studies indicated that the concept of critical flaw size and crack length as a pre-condition to unstable crack propagation was indeed

applicable to concrete. Further work by Glucklich[21] in 1962 resulted in similar conclusions and indicated that $G_c$ is a material constant. In applying fracture mechanics to a heterogeneous material such as portland cement concrete it is of course necessary to treat the material properties as average values that must vary at least on a microscopic scale.

Portland cement mortars and concretes, like metals, show values of energy required to form new fracture surface that are orders of magnitude greater than would be calculated from consideration of theoretical specific surface energy alone.

Glucklich[22] postulated that this behavior occurred in cement, a material that does not show any ductile characteristics under normal loading rates, by the mechanism of micro-cracking ahead of the main crack tip. This micro-cracking serves as an absorber of energy in an irreversible manner similar in effect to the ductile plastic straining observed in metals.

Moavenzadeh, et al[20] have shown, by microscopic observation of samples that were rejoined after fracture, sliced and polished, that this mechanism does in fact occur. This study also showed that substantial side cracking occurred and increased the required energy for crack propagation. Further increase in the energy required was attributed to the presence of aggregate particles which either required the crack to deviate around them, thus forming more fracture

surface, or, by virture of their greater fracture surface energy, absorbed more energy in the case of the crack's passing through them. This action had previously been proposed by Lott and Kessler[23].

A good exposition of the history of the application of fracture mechanics to concrete can be found in reference (19).

Applications to Wire-reinforced Mortars

In the early 1940's, Nervi[16] announced that, by plastering several layers of wire mesh with portland cement mortar, he had created a new material that showed great strength and ductility. His explanation for the different characteristics exhibited was:

> "The concept of this material is based on the elementary and familiar observation that the elasticity of a reinforced concrete member increases in proportion to the subdivision and distribution of the reinforcement throughout the mass."[16]

He was sufficiently confident of his material (which he named "Ferro-Cemento") to start construction of two torpedo boats for the Italian Navy (which ceased operations before they were finished) and later, after the war, to construct a cargo-carrying motor-sailer of 165 tons displacement and a yacht of about 40 feet length. It is interesting to note[24] that the first patent ever issued for a process of reinforcing concrete with steel in any form was issued in the late 1840's in France for a process to be called "Ferro-Cemento" and to be applied to marine applications.

The oldest known extant structure of reinforced concrete is a boat built in 1848 and exhibited in the World's Fair in Paris in 1849. Another, a similar boat built in 1887, is still in daily use in a pond in the Amsterdam Park Zoo. Following World War II many boats have been built using similar processes, most of them in Britain, Australia and New Zealand.

Romualdi and Batson seem to have been the first[(5)] to apply the concepts of fracture mechanics to the explanation of the mechanism by which the increased elongations and flexibilities shown by Nervi's "new" material are acquired. They perceived an analogy between the action of the mesh and one of the methods used to prevent catastrophic failure of ships made of steel showing brittle fracture tendencies. This method involved the riveting of steel strakes onto the hull of the ship normal to the expected path of any tensile cracks. As a crack approached the strake, the increased strain field associated with the tip of the crack would be opposed by the strake, which acted as a line of higher-modulus material in the crack's path. The stress set up in the basic hull material in attempting to strain the strake to match the hull material opposed the stress concentration at the tip and, in effect, "pinched off" the crack. This action is depicted in figure 1. In terms of equation (6) it can be said that the strake contributes a negative value to the resultant stress intensity factor affecting the advancing crack.

In their analysis of a crack moving through a member of mesh-reinforced mortar, they visualized the interaction of a similar stress field radiating out from a wire in the path of the crack. This stress field was postulated to result from the fact that the wire, of much higher modulus than the mortar, would be expected to resist the deformation necessary to allow compatability of displacements to occur at the interface between its surface and the mortar. Assuming the bond at this interface remains intact, so that this displacement compatability is in fact obtained, and assuming that both materials exhibit monotonically positive ratios of stress to strain, a shear stress must exist over the interface and must be radiated into both wire and mortar. This shear stress field must involve a shear strain field of like orientation, which opposes the component of the strain field ahead of the crack tip in the direction parallel to the wire. This action is depicted in figure 2. The theoretical distribution of this shear stress field at the mortar-wire interface can be shown [26] to be of the form depicted in figure 3. It will be noted that this shear stress has a zero value directly in the path of the crack, reaches a maximum value near the crack path, and decreases thereafter as distance from the crack path is increased.

In their original work [5], Romualdi and Batson indicated that the effectiveness of dispersed wire reinforcing should vary as the inverse square root of the wire spacing. This is to be expected because the distance a crack radiating

from an existing flaw must travel before reaching the vicinity of a wire is linearly related to the spacing of the wires (with averaging parameters included to compensate for the spacially random occurrence of the natural flaws). See figure 4. Since the value of strain energy release rate "G" varies as the square of the crack dimensions (see equation (1), the effectiveness of the wire in preventing unstable crack propagation must vary inversely as the square root of the maximum allowed crack size at a given load, and hence as the square root of the wire spacing. This relationship was observed in the experiments reported in reference (5) and has since been verified by others as well as by later work by the original investigators who have shown the mechanism also to be effective when the continuous mesh reinforcement is replaced by short randomly oriented steel fibers.(6) (7) (8) (9) (10) (11)

Such interaction of the reinforcing material with the matrix in such a manner as to cause the matrix to alter its characteristics and support more of the load than it would if it were alone, is termed "two phase" behavior. This marks the difference between normally reinforced concrete, in which large reinforcing bars are used, and mesh or fiber reinforced concrete or mortar. In such "normally" reinforced members, the concrete or mortar is of such low and unreliable tensile strength that the tensile load-carrying ability of the concrete or mortar is of such low and unreliable tensile strength that the tensile load-carrying ability of the

concrete or mortar is usually disregarded in calculating the strength of the member. If the true two-phase behavior is realized, it is possible that the concrete or mortar may carry part or most of the tensile as well as the compressive loads.

In their pioneering work [5] Romualdi and Batson noted that their wire reinforced beams failed "in a sudden manner with no previous warning other than a mild cracking." Other investigators [12] [13] [14] have noted that their similarly reinforced beams failed under similar loadings in a different manner, wherein cracks formed from the tensile face of the beams in a stable manner, the beam failing as the individual wires were stretched and broken. While these particular beams may have shown two-phase behavior in the early stages of their loading, it is obvious from the method of failure that this condition did not obtain up to the point of ultimate failure.

In a term project report at MIT, the author [15] found a similar difference in the methods of failure. While one set of samples remained intact and the mortar unbroken (save for hairline tensile cracks) until the moment of sudden failure (which occurred at substantially greater loading than the wire could sustain alone) the other set failed in the mortar at fairly low loading and at much less load than the wires could sustain. The ultimate mehtod of failure in this case was one of stretching of the wires until they failed singly. All variables (mortar compressive

strength, cure time, absence of voids etc.) between the two sets of samples indicated that the second (low strength yield) set of samples should have been superior. They were, in fact, inferior from the standpoint of ultimate load-carrying ability and strain prior to failure.

The reasons for this difference in method of failure have not to date been explored. A possible explanation lies in a difference in blond strength existing between the steel wire and the mortar at the time of loading. In the case of reinforcing with straight wires only, the only linking between the mortar and the wire, and hence the only mechanism by which any action to prevent crack propagation can be developed by the wire reinforcement, is the shear bond at the wire-mortar interface. In the event that this bond is weak or non-existent, no two-phase behavior would be predicted by the Romualdi-Batson theory. To the extent that this bond remains intact under the increased stress due to the advancing crack the crack-arrest mechanism would be expected to exist, and two-phase behavior predicted. The mechanism of development of this bond is at present unclear and the methods of prediction or measurement are unsatisfactory, at least where small wires are concerned. Work has been done on the bond strengths of cement to steel bars of larger diameters, but most of it has been undertaken from a phenomenological standpoint.[27]

The exact nature and strength of bond on a small scale has yet to be determined. Consideration of the small

strains involved in the prevention of a propagating crack indicates that the initial bond, not that developed after some relative motion has taken place, is of primary interest. The fracture cracks observed in reference (2) were on the order of .01" wide.

Much interest has been generated in the marine field by the writings of many popular journalists regarding the work of Nervi and the subsequent construction of many successful small boats utilizing the methods he proposed. Most of the investigative work in the field undertaken by commercial interests has been concerned with allowing increased loading and/or reduced structural sections without causing tensile cracking or tensile failure in the structure. Much of this work has been concerned with the addition of epoxies, binders and "flexiblizers" to the mortar. None of it has been singularly successful. In this context, the behavior observed by the author (15) for the one set of samples appears intriguing. In a ship or boat structure the primary loads imposed on the hull are those of girder-type bending, with the greatest tensile and compressive stresses set up in the keel and deck as the vessel undergoes working in the seaway. This allows concentration of reinforcing steel at these points in a highly efficient manner. These concentrations of steel serve to carry the primary loads of concern and leave the rest of the hull to carry local and hydrostatic loadings. For this purpose, a material that will accept large loadings and strains without

breaking down is of more interest than one that will accept extreme ultimate strains without actual separation, but will crack at lower loads. Therefore the determination of the differences causing the different modes of behavior is felt to be of importance.

## MATERIALS AND PROCEDURES

### Preparation of Samples

Two main types of samples were constructed. One type consisted of small "bars" that were to be pulled in uniaxial tension and examined microscopically for evidence of differences in bond characteristics of the wire/mortar interface. The other type was a series of "dog-bone" shaped samples, half of which were treated in the neck area with a silicone oil to chemically debond the mortar from the wire surface. Standard tension and compression samples were constructed to indicate the characteristics of the mortar itself. In addition, a series of wire pullout samples were included for the purpose of obtaining gross shear bond values for the interface.

The mortar was the same mix for all samples. The mixture was as indicated below;

Type of Cement: Portland Type III (high early strength)

Type of Sand: Ottowa silica graded type C-109

Sand/Cement Ratio: 0.7

Water/Cement Ratio: 0.4

The mesh used for reinforcing consisted of 19 gage (0.042 inch diameter) steel wire, galvanized by a hot-dip process after weaving to a total thickness of 0.05 inches. Mesh size was 0.5 inches by 0.5 inches. Aside from the zinc coating, no method was used to fix the

wire at the mesh intersections. Six layers were used in all mesh-reinforced samples.

The standard samples used to determine the characteristics of the mortar were two-inch cubes, constructed and tested in compression in accordance with ASTM C-109-64[28] and tensile briquettes, constructed and tested in accordance with ASTM C-190-63[28]. These samples were repeated for all batches of mortar made.

The small tensile bars were constructed by laying up several one foot square "sandwiches" of six layers of wire mesh on a plywood backing plate that was covered with a sheet of polyethelene film for smoothness and easy release of the mortar. This mesh was then plastered by hand with the mortar mixture, massaged by hand to insure good penetration and finished off to a mortar cover depth of about 1/8 inch with a small steel trowel. Care was taken to insure that no unnecessary trowelling took place that might cause segregation of the mortar. After a one-day period of hardening in the moist room, followed by a six-day cure under water the bars were cut from the sheets by use of a diamond saw. The finished dimensions were one inch by one half inch by six inches. In order to insure proper gripping of the samples during testing without causing compression failure in the way of the grips, it was necessary to cement shims of steel onto the ends of the samples. An epoxy cement was used for

this purpose. A sketch of this type of sample is shown in figure 5.

The larger "dog-bone" mesh-reinforced samples were not cut from plates because of the difficulty of cutting the material into curved shapes with the equipment available, but were cast into shape in a mold constructed for the purpose. The samples were 14 inches long, 6 inches wide at the grips and 2 inches wide at the neck section. Finished thickness was 1/2 inch throughout. The six layers of mesh were cut to shape on a bandsaw, wired together into a sandwich and placed in the mold, where they were plastered with mortar in a manner similar to the bar samples. After an overnight hardening period in the moist room, they were unmolded and placed under water for a six-day cure. Gripping in the testing machine was accomplished by drilling the samples to accept five bolts at either end and bolting them into a set of 1/4 inch thick steel plates that were adapted to the grips on the testing machine. A photograph of such a sample in the manufactured gripping system is shown mounted in the machine in figure 6. As previously mentioned, one set (five) of these samples was treated to prevent chemical bonding between wire surface and mortar by means of coating the wire in the neck area with a silicone oil. The oil used for this purpose was "Krylon" brand spray oil, of the type normally used to prevent rust from forming on machinery exposed to the elements. Two coats were

applied, drying between coatings.

In order to determine the gross bond characteristics of the wire-mortar system, several types of pullout specimens were constructed. For this purpose, several sets of the standard tensile test briquette molds were filled with plaster-of-paris, which was allowed to harden overnight. These plaster dog-bones were then cut in half across the neck section and drilled lengthwise with a 3/32 inch drill bit. Thus prepared, each half of a plaster dog-bone was placed back into one of the molds from which it was made, where a wire was inserted into the hole with a predetermined length protruding for incasement in mortar. Mortar was then placed into the other half of the mold and was gently tamped around the protruding wire. These samples were cured to the same schedule as the dog-bone and bar specimens. These pullout specimens were made with wire depths in the mortar of 1/2, 3/4, 1 and 1 1/2 inches. Samples were also made (for the 3/4 and 1 1/2 inch sets) with cross-wires left in place in order to simulate the conditions in the mesh-reinforced samples. In addition, duplicates were made of all pullout samples with the protruding portion of the wire coated with silicone oil. A sketch of a pullout sample is shown in figure 7.

Short lengths of the wire mesh were cut for testing under tensile loading. Various lengths were cut, select-

ed to include include cross-wires in half the tests. Additionally, squares of the mesh were cut and folded to allow testing of the wires in bundles.

Testing Procedures

The compression cube samples were tested in a standard hydraulic testing machine, the loading rate being fifteen pounds per second.

All tensile testing was done on an Instron testing machine. This machine includes provisions for the use of a linear differential transformer type of extensometer that furnishes indication of the average strain over a one inch gage length. This signal can be used to drive the paper of a pen-type recorder, whose pen trace indicates the load being applied to the sample. The resulting curve of load versus strain is easily converted to a stress-strain curve. This system was used in all tensile tests conducted on reinforced mortar. Wire, pure mortar and pullout samples were tested using constant speed drive for the chart paper.

All wire samples were pulled to rupture. All reinforced mortar samples were loaded until significant disruption of the mortar was noted or until maximum load was reached, whichever occurred first. Pullout samples were loaded until wire rupture or pullout of at least 1/8 inch was noted, whichever occurred first. Pure mortar samples were loaded to rupture and separation.

Examination of Samples

The bar-type samples were sectioned where possible to allow detailed examination under the optical and scanning electron microscopes. The first "roughing out" of the sample area to be examined was accomplished by means of a table saw fitted with a diamond blade. A finer cut was then taken with a very thin diamond saw blade mounted on a hydraulic ram-type shaper that allows precision control of the position and depth of cut. With this system it was possible to cut the sample very closely behind the fracture surface without disrupting the mortar. Cuts were taken both along the axis and across the samples. For comparison purposes, a sample of unstrained mesh-reinforced mortar was cut in the same manner. After cutting, the samples were sanded lightly on a flat surface using silicon carbide paper. This was necessary in order to remove the particles of wire that were smeared over the surface by the action of the cutting blade.

Examination of the surfaces revealed by the cutting procedure, as well as the fracture surfaces was most easily carried out visually using the stereo microscope. The great depth of field of this instrument made it possible to follow a crack across a fracture surface and to examine the nature of the mortar around a wire with ease. Most optical examination was there-

for carried out on this instrument. Unfortunately,no provision is made for photography on the stereo microscope, such photography necessarily being done on the Reichart optical reflected-light instrument utilizing an integrally-mounted Polaroid camera. The depth of field is very poor when compared to that of the stereo microscope, with the result that examination of fracture surfaces is greatly hampered. To a lesser degree the examination of the mortar-wire interface of a cut specimen is rendered difficult because the action of sanding the surface cuts the mortar down slightly below the surface of the wire, which is of course a harder material.

Several samples were shadowed on the fracture surfaces for examination in the scanning electron microscope. This machine has a great depth of field combined with the ability to "zoom in" to an area of interest at high magnification.

The large "dog-bone" samples were not examined microscopically.

## RESULTS

### Wire Break Tests

A total of 26 wires were broken individually. Five bundles of four wires each were also broken. The average breaking strength per wire was found to be 140 lbs., the maximum deviation from this value being 4 lbs. above and below. The bundle strength was found to be 560 lbs., the sum of the expected wire strengths.

### Pure Mortar Tests

A total of nine compressive and nine tensile tests were made using pure mortar. The results are found in Table 1.

### Pullout Tests

Of all the pullout tests made, only the samples coated with the silicone oil and with a wire depth of 1/2 inch pulled out. All others broke off at or above the surface of the mortar and broke at the predicted wire strength of about 140 lbs. The three samples that pulled out did so at values of 32 lbs., 40 lbs., and 96 lbs.

### Bar Tensile Tests

A total of eight bar samples of mesh-reinforced mortar were tested. The results are shown on the load-strain curve in figure 8. The samples contained different total numbers of wires oriented in the direction of applied tensile load, but consideration of this is

deferred to the section dealing with the discussion and analysis of results.

Dog-Bone Tensile Tests

A total of twelve dog-bone samples of mesh-reinforced mortar were tested, five of which had been treated with silicone oil to prevent effective bond between the mortar and the mesh. The results of these tests are presented in figures 9 and 10. The samples treated with oil are shown separately from the "normal" samples.

## DISCUSSION OF RESULTS

Two separate and distinct modes of failure were observed during the tensile testing of the small bar-type specimens. See figure 11. One type of failure consisted of the formation of small, short hairline cracks at a tensile strain of about 0.1%, followed by their continuous growth until the mortar was cracked through the sample in several parallel planes. Further extension resulted in complete disruption of the mortar over the length of the sample until the wire, loaded to its ultimate load-carrying ability, failed in a ductile manner. The tensile cracks remained open after failure. The other observed mode of failure was dramatically different. Although the first small tensile cracks appeared on the surface of the mortar at the same strain, they did not widen or lengthen appreciably as the sample was loaded further. Instead, more cracks of similar nature were formed as loading continued. At a load approaching the maximum load-carrying ability of the wire mesh, these cracks were seen to begin to link up, and the sample failed suddenly and completely at that point. Upon the release of the load by failure of the sample, most of the tensile cracks closed and were not visible to the naked eye. After being loaded to strains of 0.3%, the intact portions of these samples did not exhibit any residual cracking after release of the load.

The samples contained slightly different numbers of wires oriented in the direction of the applied tensile stress, so it is necessary to normalize the results in order to evaluate the performance of the samples. This normalization has been carried out in table 2. It is noted that strengths slightly in excess of predicted wire load-carrying ability were obtained in all cases wherein the sample failed in a sudden manner. This slight increase in load-carrying ability was accompanied in all cases by apparent structural integrity of the remaining portion of the sample, whereas all the slow failures resulted in extensive disruption of the mortar over the test section. This difference is clearly shown in figure 11. Most of the samples that suffered slow failure were not pulled to complete separation because the extensive mortar disruption made sectioning for microscopic examination difficult if not impossible.

Because of the mode of failure, no fracture surface was available for examination from those samples that failed in a slow manner. The fracture surface from one of the suddenly-failing samples is shown in figure 12. These photographs were obtained on a scanning electron microscope and were taken at a magnification of thirty diameters. The photograph in figure 12(a) shows a portion of the surface containing a protruding wire that was oriented in the direction of applied stress. It is

noted that the wire has failed in a "cup and cone" manner associated with ductile failure in steel. The zinc galvanizing coating is seen to be pulled away from the steel over a large portion of the necked area of the wire, as well as being separated from the mortar. The longitudinal scratches in the necked area of the wire indicate that substantual relative motion took place between steel and zinc prior to separation. The photograph shown in figure 12(b) is of a portion of the fracture surface that includes a shallow hole where one of the wires aligned in the direction of applied stress broke off below the surface. The characteristics of the mortar fracture surface are clearly shown here. The appearance of the surface indicates that the fracture surface passed mainly through the paste phase of the mortar, passing around rather than through the aggregate particles. The small crack in the surface in the upper left-hand portion of the picture is located directly over a wire that runs across the sample. The wire is approximately 1/8 inch below the surface. These pictures are typical of all the fracture surfaces examined under the stereo microscope.

After cutting with a diamond saw under a water spray, examination of the samples revealed several interesting crack patterns. In both types of samples, examination of the area surrounding the wires that were oriented in the direction of applied tensile stress revealed no ap-

parent movement between the mortar and the wire. It was impossible to determine optically whether the bond between the two had been broken. Only very small cracks were observed, having no apparent preferred orientation. These cracks had the appearance of being the intersections of crack planes with the cut surface at very shallow angles. This is to be expected since tensile cracks should be initiated and should propagate in directions perpendicular to the axis of applied tensile stress[(1)]. The appearance of the samples observed in this direction is shown in figure 13. This assumption as to the orientation of the crack planes was substantiated by wetting the sample slightly and observing the area around a crack as it dried. The cracks were then seen to have a "feather edge" where their planes of propagation intersected the cut plane at a very acute angle. This condition existed in both types of samples, those that failed slowly and those that failed suddenly.

Observation of the samples in a direction perpendicular to the direction of applied stress revealed greater differences in the samples, particularly in the areas immediately surrounding the wires.

For the case of the samples that failed in a sudden manner, it was found that the cracks around the wires intersected the wire-mortar interface at points in the direction of the applied tensile stress. After

leaving the wire surface they then bent to align themselves perpindicular to the axis of applied stress. This is shown in the sketch in figure 14 and in the photographs in figures 15, 16 and 17. The cracking in the mortar away from the interface areas was manifold and the cracks were small. This is shown in figure 17(b).

For the case of the samples that failed slowly, the cracks in the mortar phase were fewer but were larger and more well-defined. They intersected the wire-mortar interface at points oriented ninety degrees to the axis of applied stress. In these cases the mortar was seen to be displaced from the wire over a significant portion of the interface, the crack continuing into the mortar from either end of the broken bond area. This action is shown in the sketch in figure 18 and in the photographs in figures 19, 20 and 21. The appearance of the mortar away from the wire is shown in figure 21(b).

This difference in cracking patterns can be explained by the theory of elastic inclusions in an elastic matrix developed by Goodier[30]. An indication of the positions and magnitudes of local high stresses can be obtained for the case of a cylindrical inclusion that is very stiff compared to the matrix and one that has a xero modulus of elasticity (a hole). Plotting of his equations[30] indicates that the points of high circumferential tensile stress occur in the direction of applied tensile

stress in the case of a stiff inclusion and in directions perpindicular to the applied tensile stress in the case of a very soft inclusion. The case of the hole in the infinite plate is well known, wherein the points of maximum tensile stress are at the sides and are of magnitude three times as great as the applied stress.(31) The magnitude of the stress concentration for the stiff inclusion is somewhat less than half that for the hole(30) and is ninety degrees away from it in position on the interface boundary. See the sketch in figure 22.

In our case of mesh-reinforced mortar, we have many cylindrical inclusions oriented across the applied tensile stress field, these inclusions being the cross-wires in the reinforcement. The wire has a modulus of elasticity that is approximately ten times that of the mortar, so they should be expected to act as stiff inclusions if they are well bonded to the mortar. If, on the other hand, they possess poor bond or no bond, so that little or no traction can be developed between wire and mortar, they would appear as low-modulus inclusions (holes) to developing cracks.

If high bond is developed, we would expect the cracks at the interface to start at points of maximum circumferential tensile stress, i.e. at points oriented in the direction of applied tensile stress. The magnitude of the

maximum circumferential tensile stress is low compared to the case of the hole, so the formation of these cracks should be delayed to higher imposed stresses. The direction of these cracks at initiation is in the direction of the applied stress field, so the component of the crack dimension across the applied stress field is small. In order to develope a large characteristic dimension in the Irwin criterion for unstable crack propagation (equation 2) the crack must change its course and propagate some finite distance in a direction across the applied tensile field, absorbing energy in the process. These cracks can be said to contribute a low positive value of K in equation 6.

If, on the other hand, little or no bond is present at the interface, we would again expect the cracks to form at the points of high local circumferential tensile stress, but these points are now located across the applied tensile stress direction and these local stresses are much higher. It should be expected that cracks would form at lower applied stress levels. These cracks are oriented upon formation in the direction of easiest propagation. Furthermore, any broken bond area is in such orientation as to furnish additional characteristic length in the Irwin criterion (equation 2). It can be said that these cracks contribute an initial large positive value of K to equation 6. Thus it would be expected

that under these conditions of low bond the cross-wires in the mesh reinforcement would act as long and very potent stress raisers, giving rise to cracks oriented in the proper direction for easy propagation through the sample. If the bond were good, not only would the tensile circumferential stresses induced at the surface be lower in magnitude, but any cracks formed thereby would be oriented in a less harmful direction.

Examination of the photographs in figures 15, 16, 17, 19, 20 and 21 indicates that this difference in crack patterns that would be predicted on the basis of bond strength is in fact observed in the two types of samples, the pattern expected for the case of poor bond being associated with the samples that failed slowly.

In an effort to experimentally evaluate the effect of wire-mortar bond on the fracture mode, a series of large "dog-bone" samples was constructed. These differed in makeup from the bar-type samples in that five of them were treated with silicone oil to retard the bond in the neck area. To check that this treatment did in fact result in reduced bond, some of the pullout samples were likewise treated. The results of these pullout tests confirmed that bond had been impaired.

Again, two types of failure were observed. The samples that had been treated with the oil all exhibited early tensile cracking with the cracks oriented across the sample neck in parallel planes. These cracks, as in

the case of the slowly-failing bar samples, grew with increased loading until the sample failed in a wide band over the neck area by mortar disruption. The samples that had not been "poisoned" by the oil addition did not exhibit tensile cracking until later in the loading. This tensile cracking did not result in large cracks until the sample approached destruction. The apparent difference in moduli between the two types of samples indicated in figures 9 and 10 is due to the measurement system used to detect strain. It was noted that in the case of the samples that had been treated with oil, little strain was indicated until a tensile crack formed between the attachment points of the extensometer, at which time the indicated strain would increase at a rapid rate. The Instron trace was marked at equal intervals of crosshead displacement, so it was possible to replot figures 9 and 10 on the basis of load versus crosshead displacement. This has been done in figure 23. It is noted that the slope of these two curves is similar over the middle portion of the range, even though the traces chosen for replotting were of different forms originally. The difference in appearance between the two types of samples after failure is shown in figure 24. Failure of the "normal" samples took place in one small area of the sample only, the remaining small tensile cracks closing up and becoming invisible after failure. Most tensile cracks

remained open in the case of the oil-treated samples after the load was removed. Some disruption of the mortar was noted in areas remote from the failure plane in the untreated samples when loading was prolonged past the point of maximum load-carrying ability, but none was noted prior to this point.

The early and parallel formation of tensile cracks that grew with loading observed in the case of the samples with impaired bond strength, coupled with the lack of such cracks in the normal samples indicates that the bond is of great importance in determining the mode of failure. The detailed examination of the small tensile specimens and the prediction of Goodier's equations indicate that the bond on the cross-wires is of particular importance.

The failure of other investigators[7][8][9][12][13][14] to observe the sudden mode of failure obtained by Romualdi[5] and the author[15] may have been due to this lack of bond on the cross-wires. Most of Romualdi's work with continuous wires[5] has been concerned with wires running only in one direction, while the workers with chopped wires[7][8][9][10][11] have usually come to the conclusion that longer wires than were used in their investigations were needed to prevent premature failure and pullout. No analysis of the effect of the wires oriented across the axis of applied stress has been attempted in these investigations. The effect of stress concentrations

caused by poor bond on these mis-orientd wires may well have shadowed their results.

CONCLUSIONS

The bond strength at the wire-mortar interface has been shown to be of great importance in preventing premature failure and non-reversible tensile cracking in mesh-reinforced portland cement mortar. The lack of such bond on wires running across a tensile stress field results in those wires acting in the manner of very potent stress raisers, tending to cause cracks that are oriented upon initiation in the proper dirction for easy propagation across the structure. In the event that good bond is present at that interface, these cross-wires act as stiff inclusions. Although these stiff inclusions are also stress raisers, the stress concentrations induced are much less severe and they are induced at such points that any tensile cracks initiated by them are not initially oriented in a direction for easy propagation. The entirely different effect of the cross-wires, caused by differences in bond on them, would indicate that there should be no gradual variation of fracture toughness of the material with gradually varying bond strength, but rather some threshold value of bond strength would cause rapid variation of fracture toughness.as it was exceeded.

## SUGGESTIONS FOR FURTHER WORK

It is felt that the mechanism presented herein concerning the crack-initiating action of the cross-wires in a mesh-reinforced mortar system is worthy of further investigation. The conducting of tests in which tensile specimens had only the cross-wires "poisoned" to inhibit bond would seem to be a next step. The use of photoelastic methods to determine the threshold value of bond required to cause crack initiation to take place at various points around a cylindrical inclusion as a function of the elastic moduli and Poisson's ratios of the materials would allow prediction of the bond strengths necessary to prevent premature failure.

LIST OF REFERENCES

1. GRIFFITH, A. A. "The Phenomenon of Rupture and Flow in Solids" Philosophical Transactions, Royal Society of London A-221, 1921 p.163

2. GRIFFITH, A. A. "Theory of Rupture" Proceedings, First International Congress of Applied Mechanics September, 1924 pp.55-63

3. IRWIN, G. R. Journal of Applied Mechanics, Vol. 24 1957, p.361

4. IRWIN, G. R. Journal of Applied Mechanics, 1939 p.49

5. ROMUALDI, J. P. and BATSON, G. B. "Behavior of Reinforced Concrete Beams With Closely Spaced Reinforcement" Journal of the American Concrete Institute, Proceedings Vol.60, June 1963 pp.775-789

6. ROMUALDI, J. P. and BATSON, G. B. "Mechanics of Crack Arrest in Concrete" Proceedings, A.S.C.E. Vol.89 EM3 June, 1963 pp. 147-168

7. McKENNEY, J. L. jr. "Tensile Strength of Steel Fiber Reinforced Concrete" Unpublished Master's Thesis, Clarkson College of Technology May, 1964

8. BAJAN, R. L. jr. "Strength of Steel Fiber Reinfoced Concrete with Aggregate" Unpublished Master's Thesis, Clarkson College of Technology June, 1965

9. BAILEY, L. E. "Fatigue Strength of Steel Fiber Reinforced Concrete" Unpublished Master's Thesis

Clarkson College of Technology October, 1966

10. ROMUALDI, J. P. and MANDEL, J. A. "Tensile Strength of Concrete Affected by Uniformly Distributed and Closely Spaced Short Lengths of Wire Reinforcement" _Journal of the American Concrete Institute, Proceedings_ Vol.61 June, 1963

11. ROMUALDI, J. P., RAMEY, M., and SANDAY, S. C. "Prevention and Control of Cracking by Use of Short Random Fibers" _A.C.I. Publication S.P.-20_

12. COLLEN, L. D. G. and KIRWIN, R. W. "Some Notes on The Characteristics of Ferro-Cement" _Civil Engineering and Public Works Review_ February, 1959 pp. 195-196

13. COLLEN, L. D. G. "Some Experiments in Design and Construction with Ferro-Cement" The Institution of Civil Engineers of Ireland, paper presented at 4 January 1960 general meeting

14. KELLEY, A. M. and MOUAT, T. W. "Ferro-Cement as a Fishing Vessel Construction Material" Conference on Fishing Vessel Construction Materials, paper presented at October, 1968 conference, Montreal, Canada, Federal-Provincial Atlantic Fishing Committee

15. COLLINS, J. F. "Tensile Strength of Mesh-Reinforced Mortar" Unpublished term report, Massachussetts Institute of Technology, May, 1968

16. NERVI, P. L. "Ferro-Cement, Its Characteristics and Potentialities" C.A.C.A. London, Translation number 60, 1959

17. KAPLAN, M. F. "Crack Propagation and the Fracture of Concrete" Journal of the American Concrete Institute, Proceedings Vol.58, No.5 Nov. 1961 pp. 591-610

18. KAPLAN, M. F. "The Application of Fracture Mechanics to Concrte" International Conference on the Structure of Concrete, London Paper D-1 Sep.1965

19. WELCH, G. B. and HAISMAN, B. "The Application of Fracture Mechanics to Concrete" UNICIV Report No.1.12 University of New South Wales, Kensington, N.S.W., Australia

20. MOAVENZADEH, F., KUGUEL, R. and KEAT, L. B. "Fracture of Concrete" M.I.T. Dept. of Civil Engineering Report R68-5 March, 1968

21. GLUCKLICH, J. "The Flexural, Static and Fatigue Strength of Portland Cement Mortar" Urbana, University of Illinois, T.A.M. Report No.622 1962

22. GLUCKLICH, J. "Fracture of Plain Concrete" Journal of the Engineering Mechanics Division, Proceedings A.S.C.E. Vol.89 No. E.M.6 pp. 127-138 1963

23. LOTT. G. L. and KESSLER, C. E. "Crack Propagation in Plain Concrete" Urbana, University of Illinois T.A.M. Report No. 648, 1964

24. FREEMAN, J. E. "Developements of Concrete Ships and Barges" Journal of the American Concrete Insstitute, Proceedings Vol.14 1918

25. IRWIN, G. R. "Analysis of Stresses and Strains Near the End of a Crack Traversing a Plate" Journal of Applied Mechanics Vol.24, 1957 p.361

26. CORNELL, M. R. "Bond Stresses Between Steel Fibers and Concrete as Effected (sic) by the Modulus of Elasticity and Poisson's Ratio of the Materials" Unplublished Master's Thesis, Clarkson College of Technology, September, 1966

27. ALEXANDER, K. M. "Mechanics of Shear Failure at the Steel-Cement and Aggregate-Cement Interface" International Conference on Structures, Solid Mechanics and Engineering Design in Civil Engineering Materials, paper No. 32, University of Southampton, 21-25 March, 1969

28. A.S.T.M. Standards, 1963 American Society for Testing and Materials 1963

29. COLLINS, J. F. and CLAMAN, J. S. "Ferro-Cement for Marine Applications, An Engineering Evaluation" New England Section, Society of Naval Architects and Marine Engineers, paper presented at March, 1969 meeting

30. GOODIER, J. N. "Concentrations of Stress around Sperical and Cylindrical Inclusions and Flaws"

Journal of Applied Mechanics Vol.55, 1933 pp.39-44

31. TIMOSHENKO, S. Theory of Elasticity 1934

| Compression | | | Tension | | |
|---|---|---|---|---|---|
| Sample | Load | Stress | Sample | Load | Stress |
| 1 | 37500 | 9345 | 1 | 850 | 850 |
| 2 | 41500 | 10375 | 2 | 846 | 846 |
| 3 | 41000 | 10250 | 3 | 849 | 849 |
| 4 | 39000 | 9750 | 4 | 854 | 854 |
| 5 | 41250 | 10312 | 5 | 850 | 850 |
| 6 | 36000 | 8500 | 6 | 852 | 852 |
| 7 | 40500 | 10125 | 7 | 831 | 831 |
| 8 | 42000 | 10500 | 8 | 844 | 844 |
| 9 | 44500 | 11125 | 9 | 853 | 853 |

Table 1.

Results of Pure Mortar Compression and Tension Tests

| Sample | Type of Failure | Maximum Load (lbs) A | Mesh Load-Carrying Ability (lbs) B | A/B |
|---|---|---|---|---|
| A | Sudden | 1700 | 1540 | 1.11 |
| B | Slow | 1330 | 1400 | 0.95 |
| C | Slow | (broke up by improper isertion in grips) | | |
| D | Slow | 1530 | 1540 | 0.99 |
| E | Sudden | 1750 | 1540 | 1.14 |
| F | Slow | 1400 | 1540 | 0.91 |
| G | Slow | 1350 | 1540 | 0.88 |
| H | Slow | 1400 | 1540 | 0.91 |
| I | Slow | 1275 | 1400 | 0.91 |
| J | Sudden | 2200 | 1821 | 1.21 |

Table 2.

Normalized Load-Carrying Ability of Bar Samples

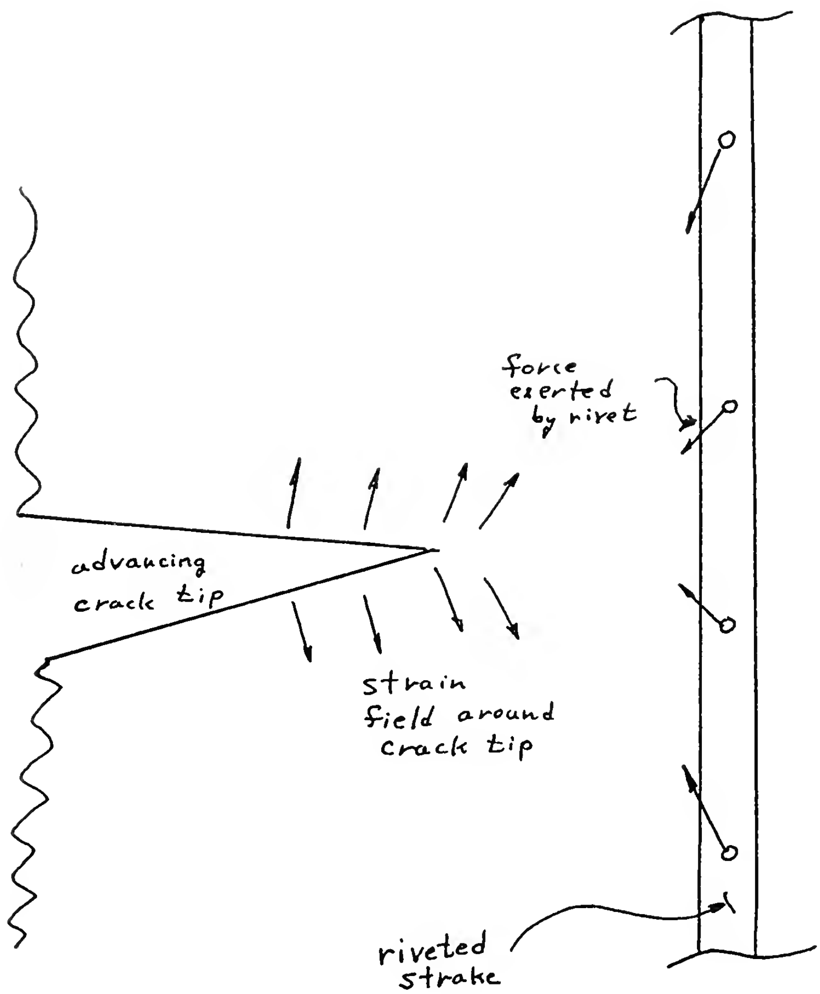

force
exerted
by rivet
advancing
crack tip
strain
field around
crack tip
riveted
strake

figure 1

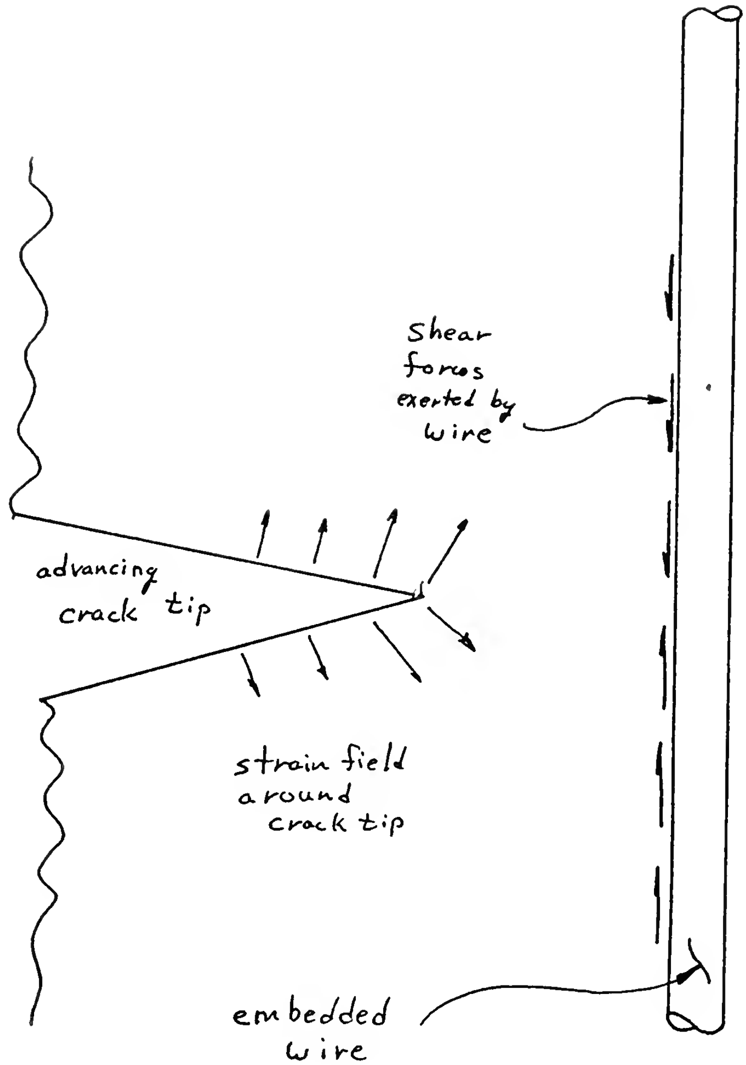

shear forces exerted by wire
advancing crack tip
strain field around crack tip
embedded wire

figure 2

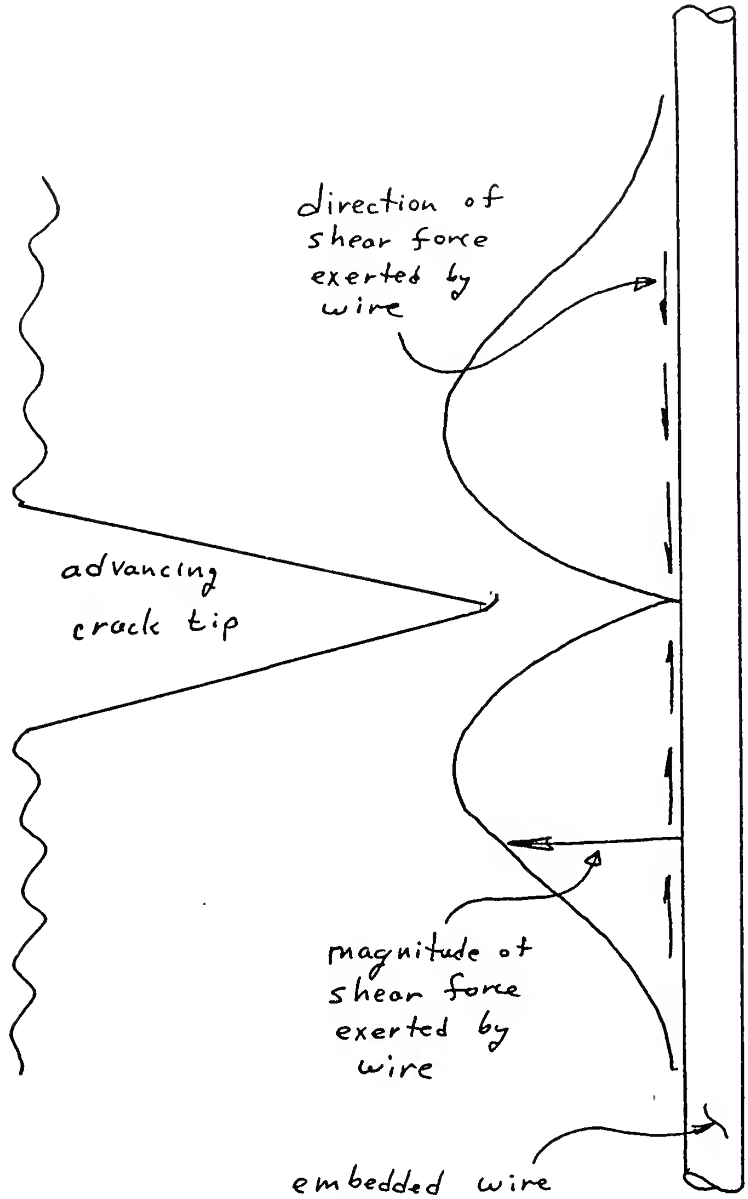

direction of
shear force
exerted by
wire
advancing
crack tip
magnitude of
shear force
exerted by
wire
embedded wire

figure 3

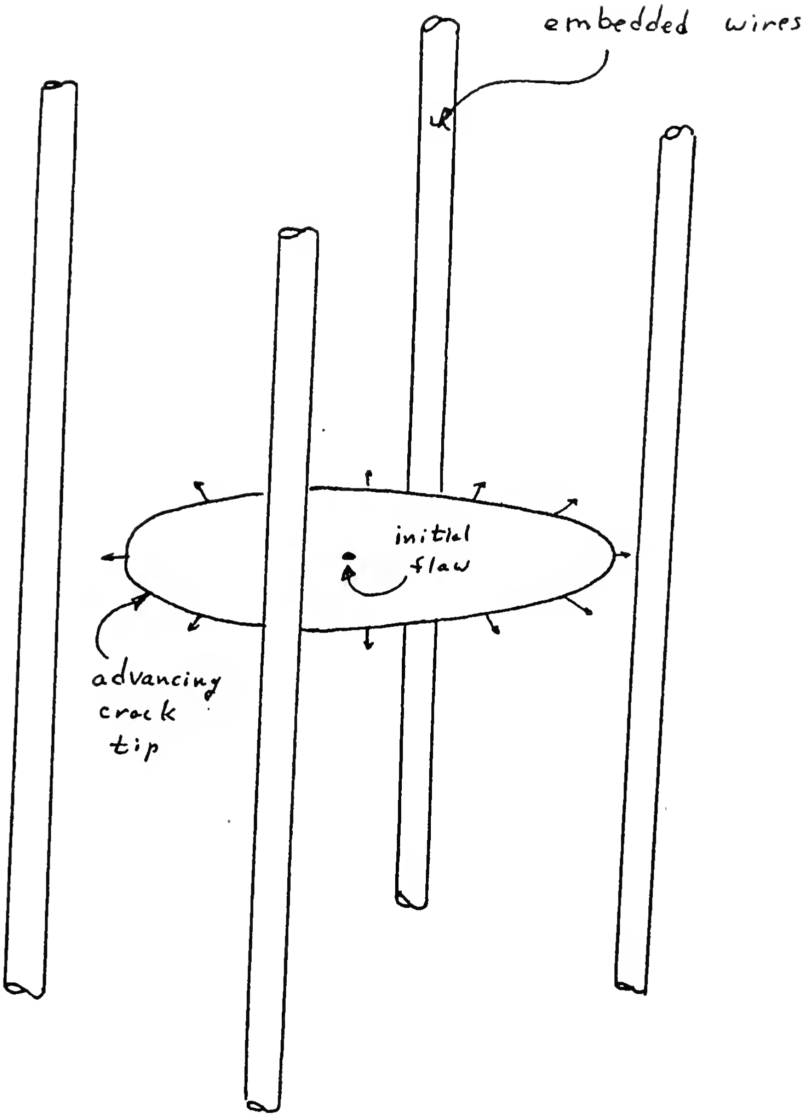



figure 4

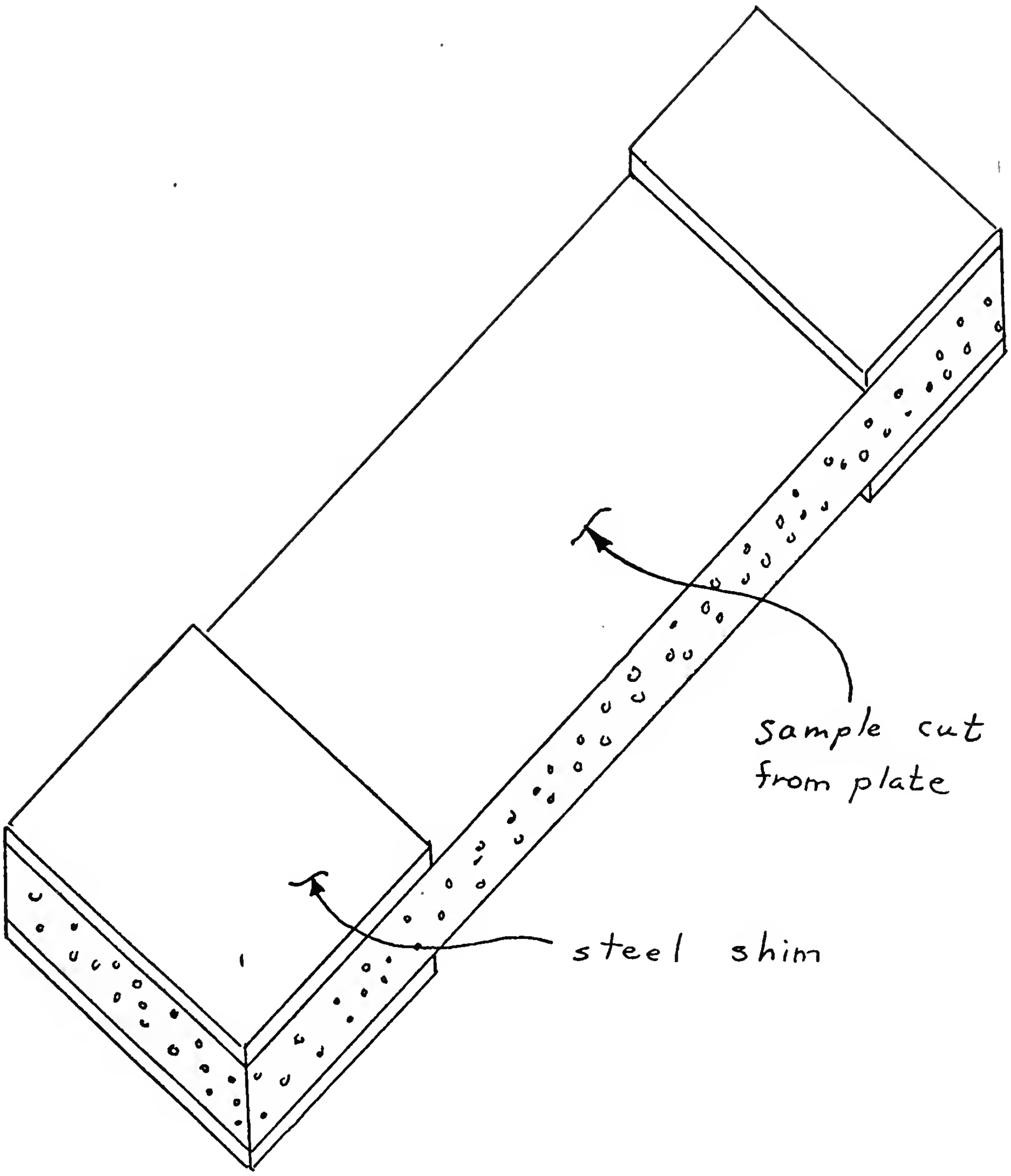



figure 5

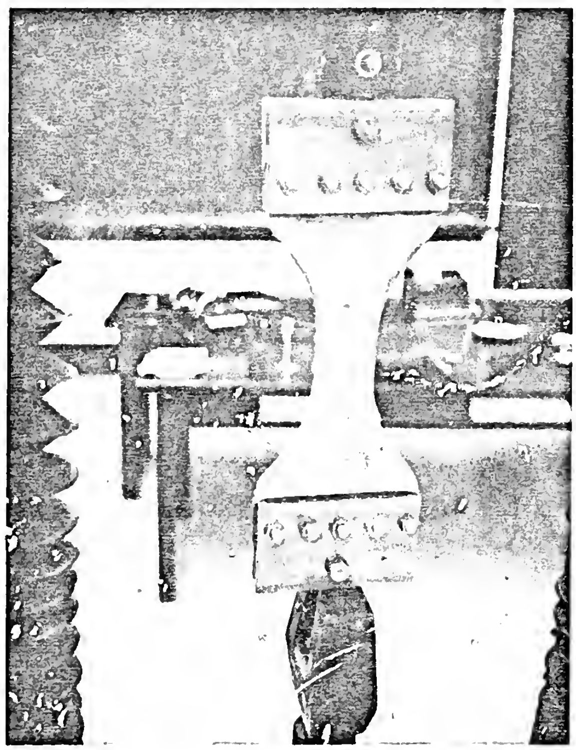


figure 6

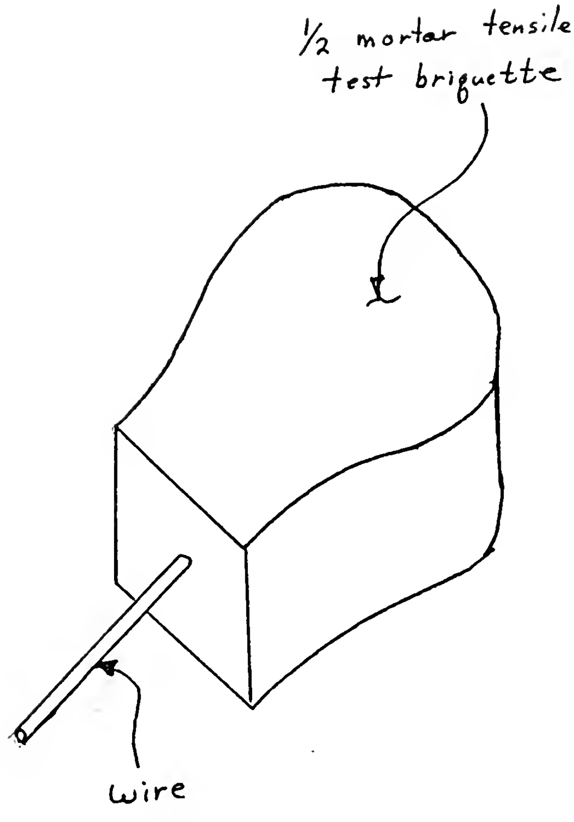



figure 7

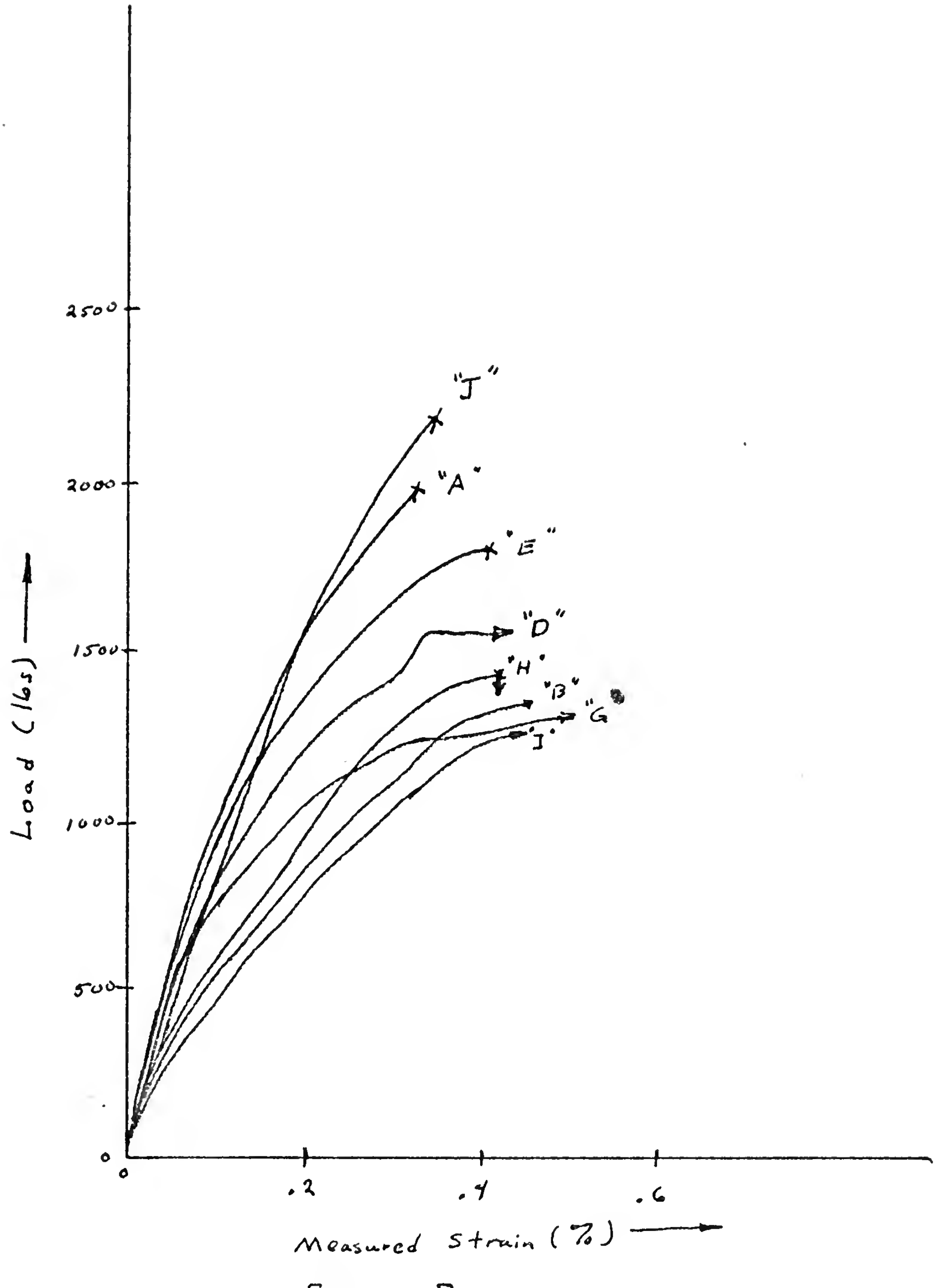

"J"
"A"
"E"
"D"
"H"
"B"
"G"
"I"
2500
2000
1500
1000
500
0
Load (lbs)
0
.2
.4
.6
Measured Strain (%)

figure 8

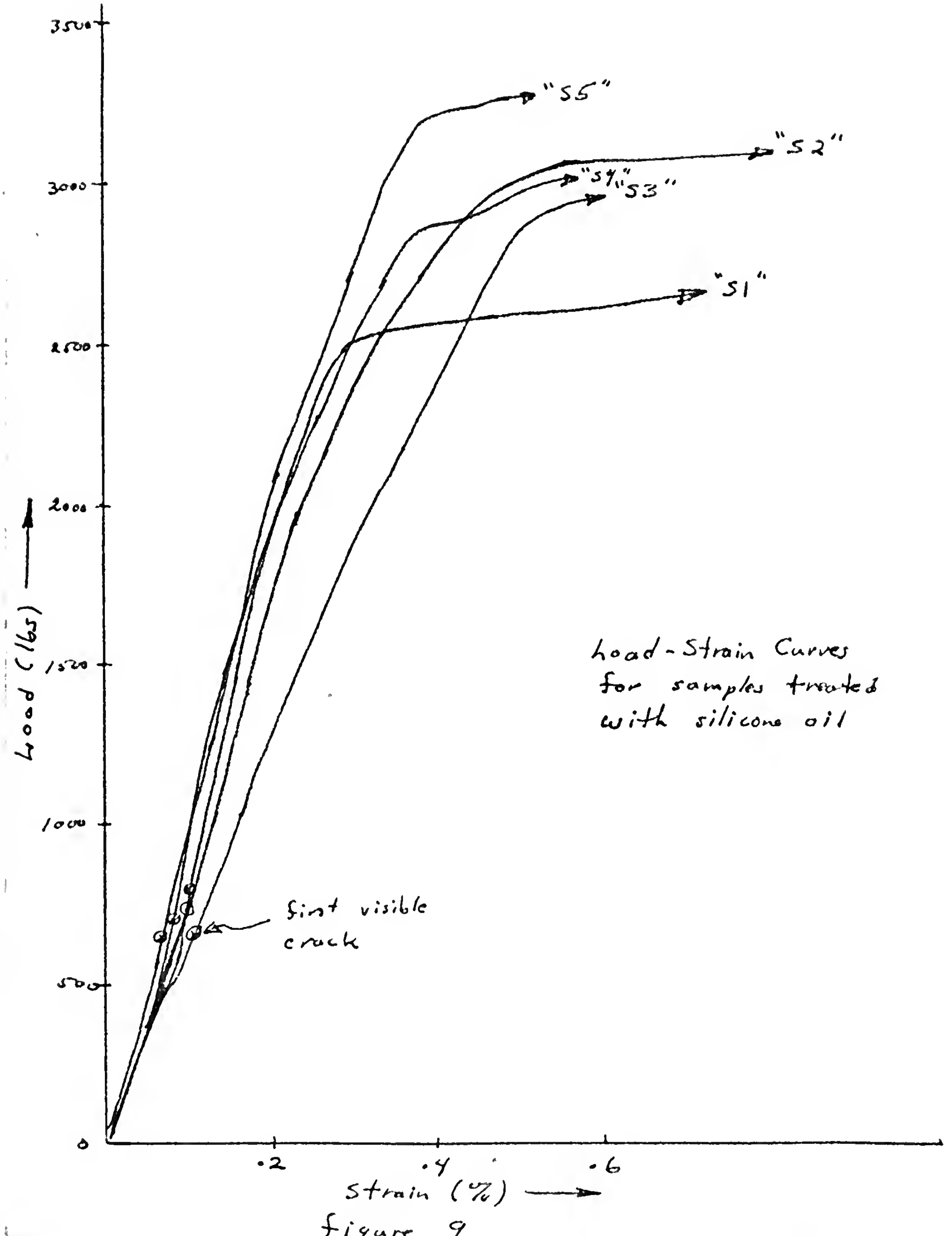
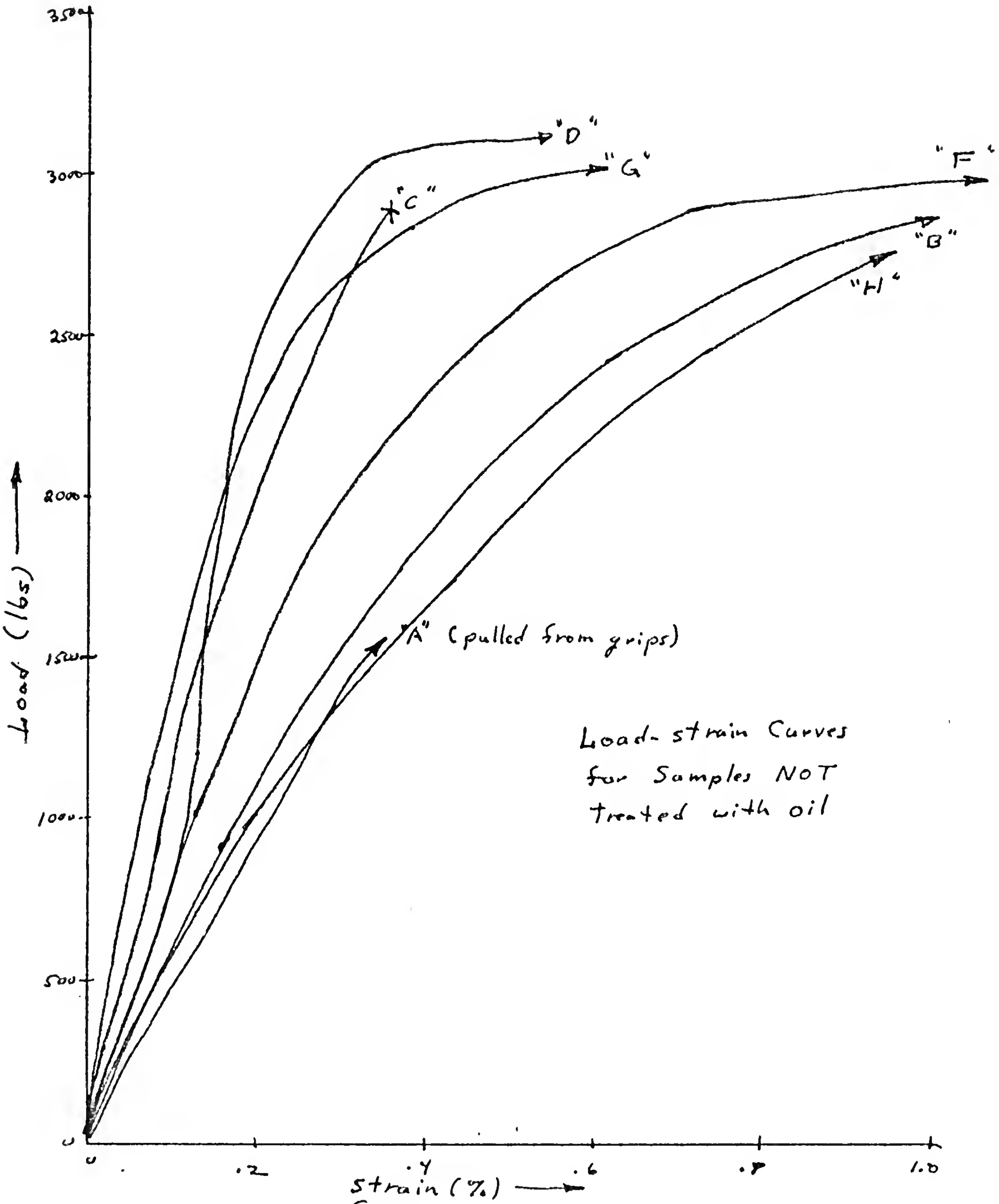

3500
3000
2500
2000
1500
1000
500
0
Load (lbs)
.2
.4
.6
Strain (%)
"S5"
"S2"
"S4"
"S3"
"S1"
first visible crack
Load-Strain Curves for samples treated with silicone oil

figure 9



3500
3000
2500
2000
1500
1000
500
0
Load (lbs)
"D"
"G"
"C"
"F"
"B"
"H"
"A" (pulled from grips)
Load-strain Curves for Samples NOT treated with oil
0
.2
.4
.6
.8
1.0
strain (%)


figure 10

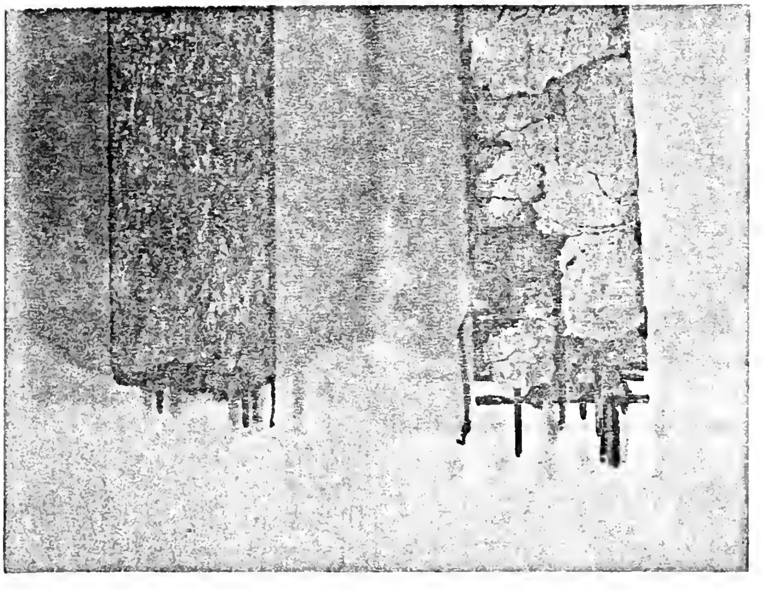
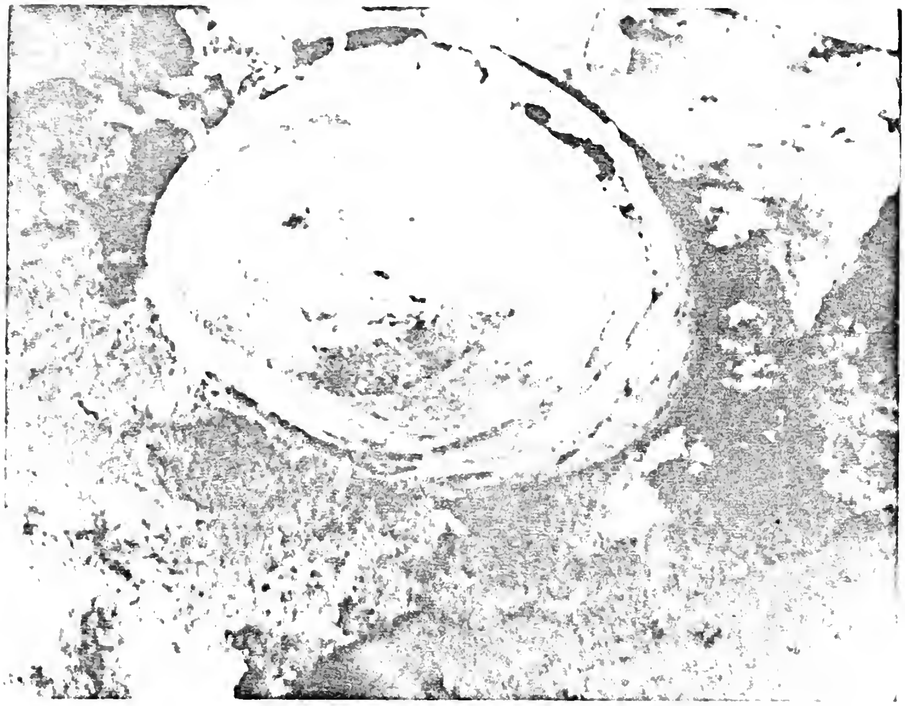
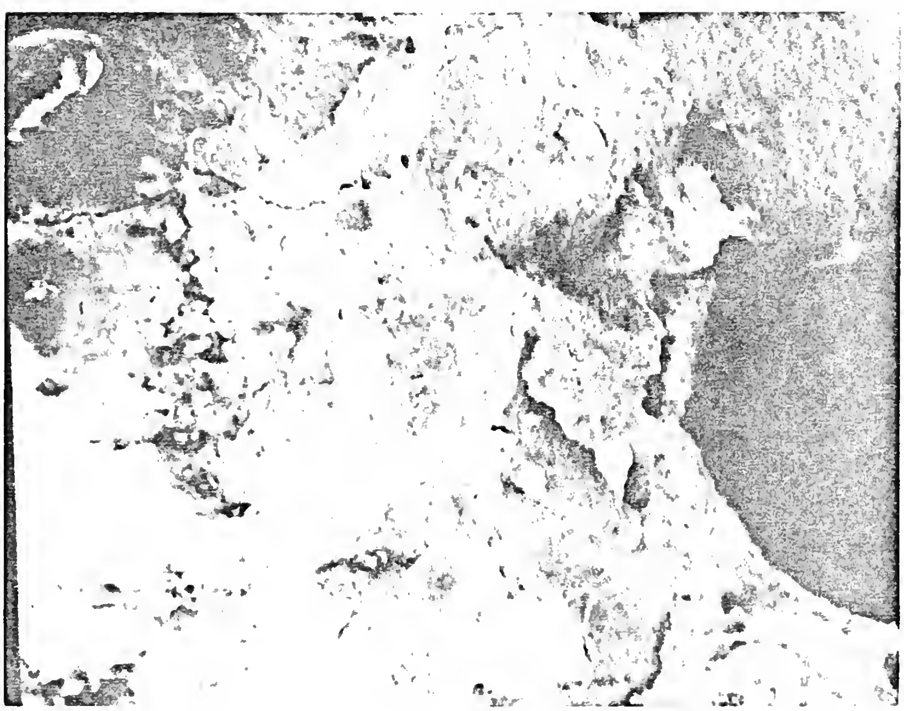


figure 11

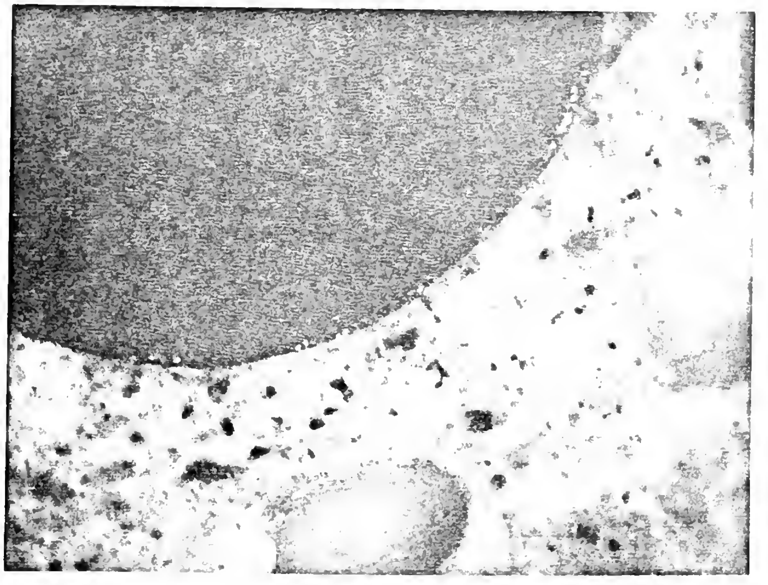


12(a) (X30)

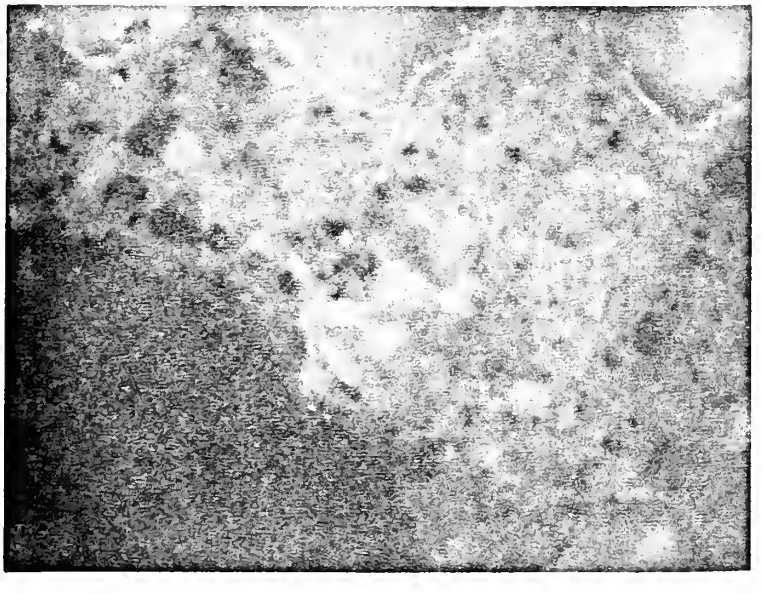


12(b) (X30)

figure 12



(a) (x44)



(b) (x44)

figure 13

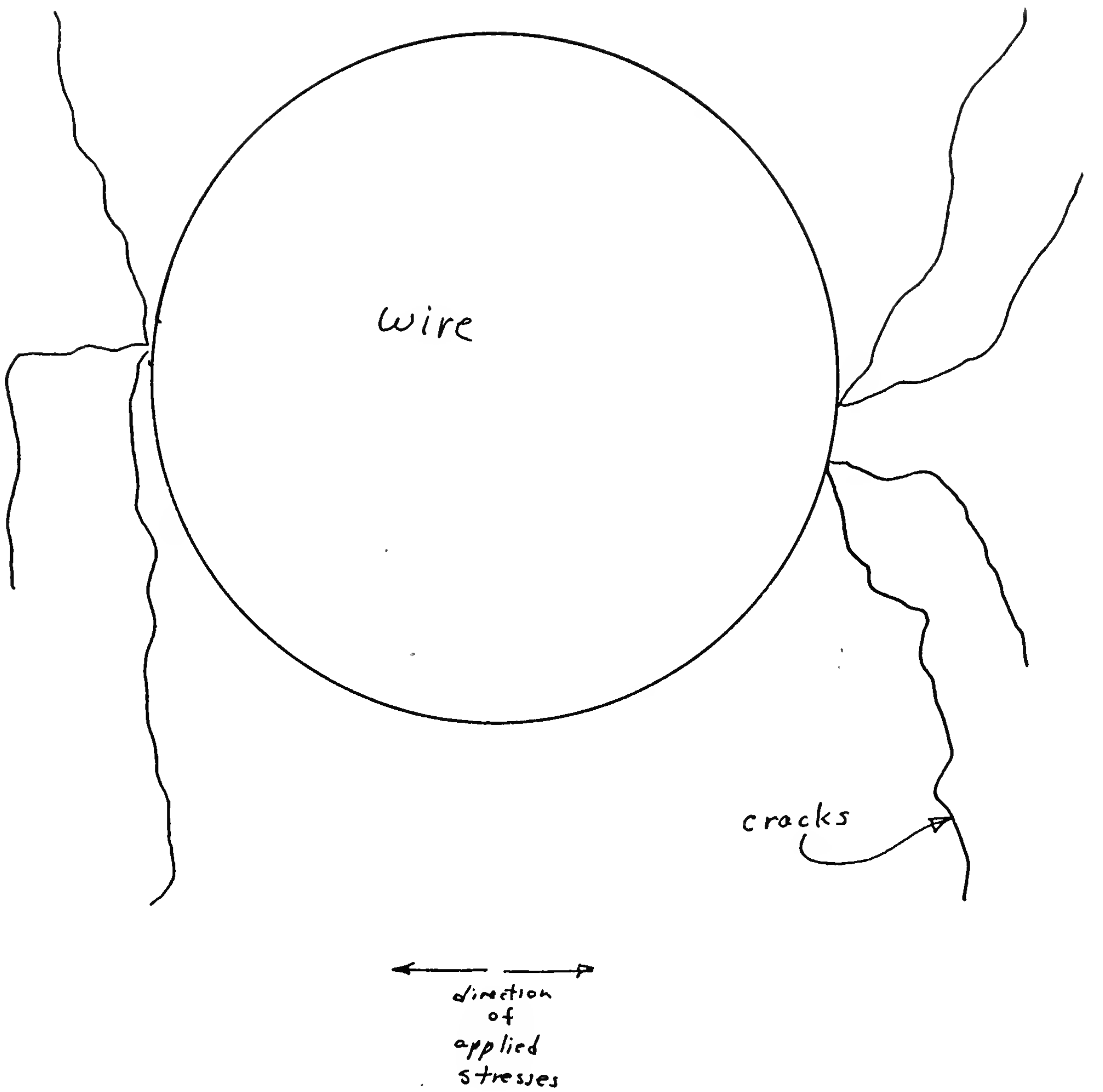
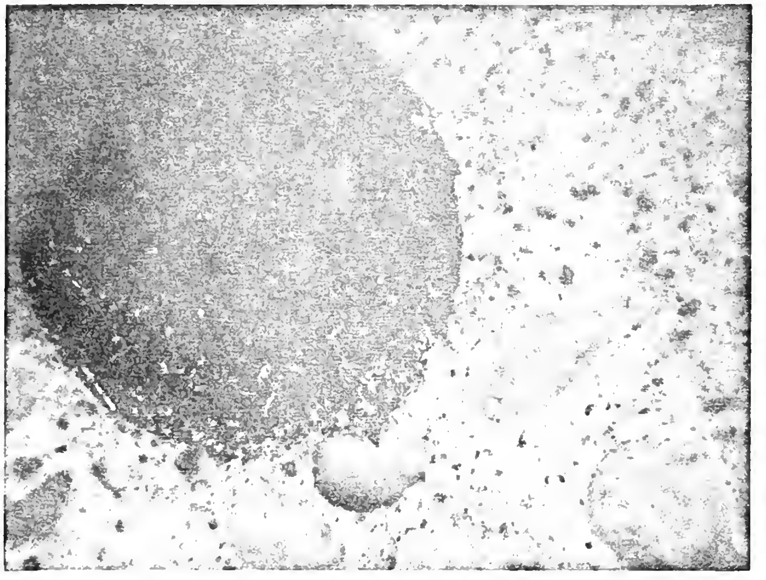
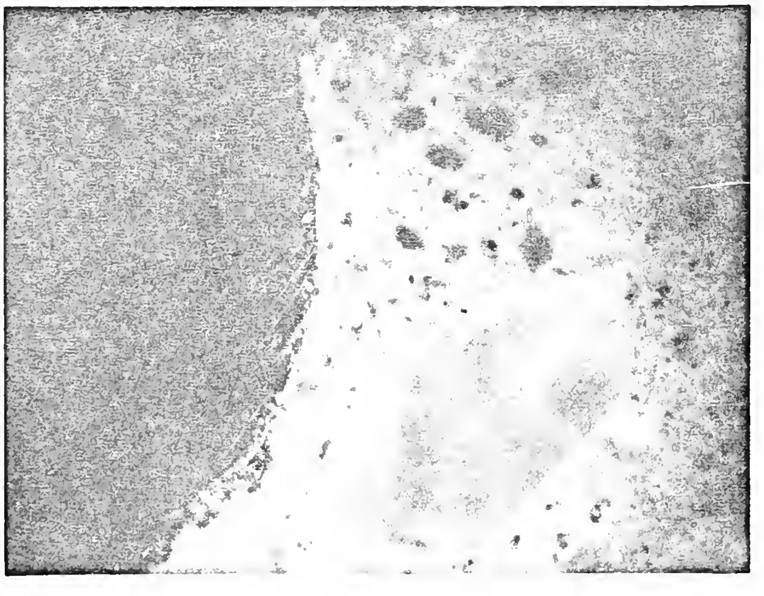



figure 14



tensile stress

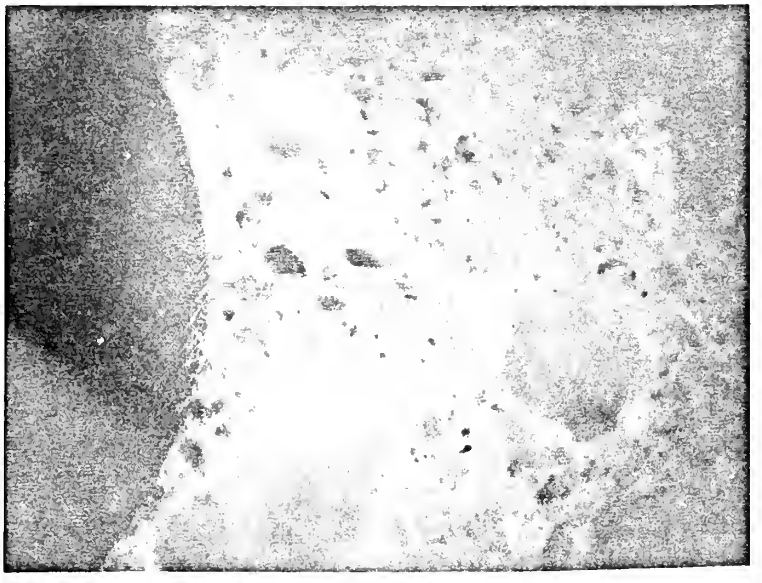
a. (x22)



tensile stress

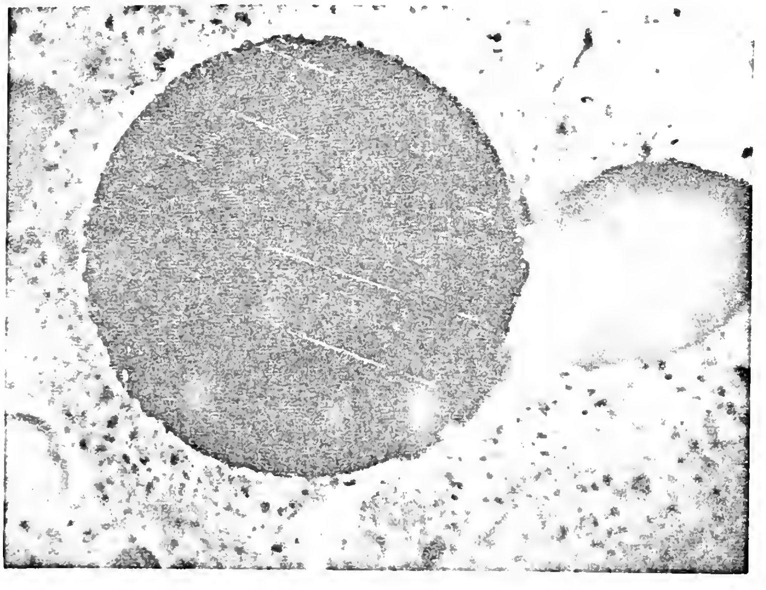
b. (x 44)

figure 15



→ tensile stress

a. (X 22)



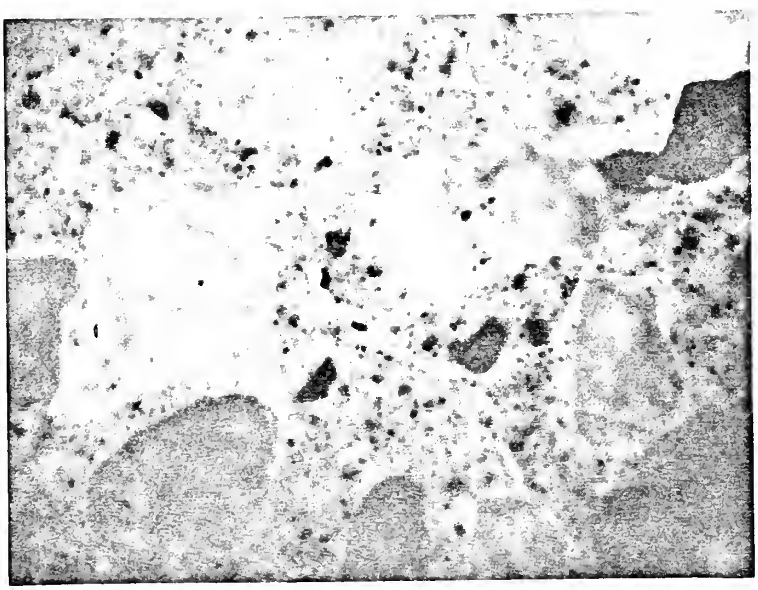
→ tensile stress

b. (X44)

figure 16

tensile stress

a (x44)



tensile stress

b (x44)

figure 17

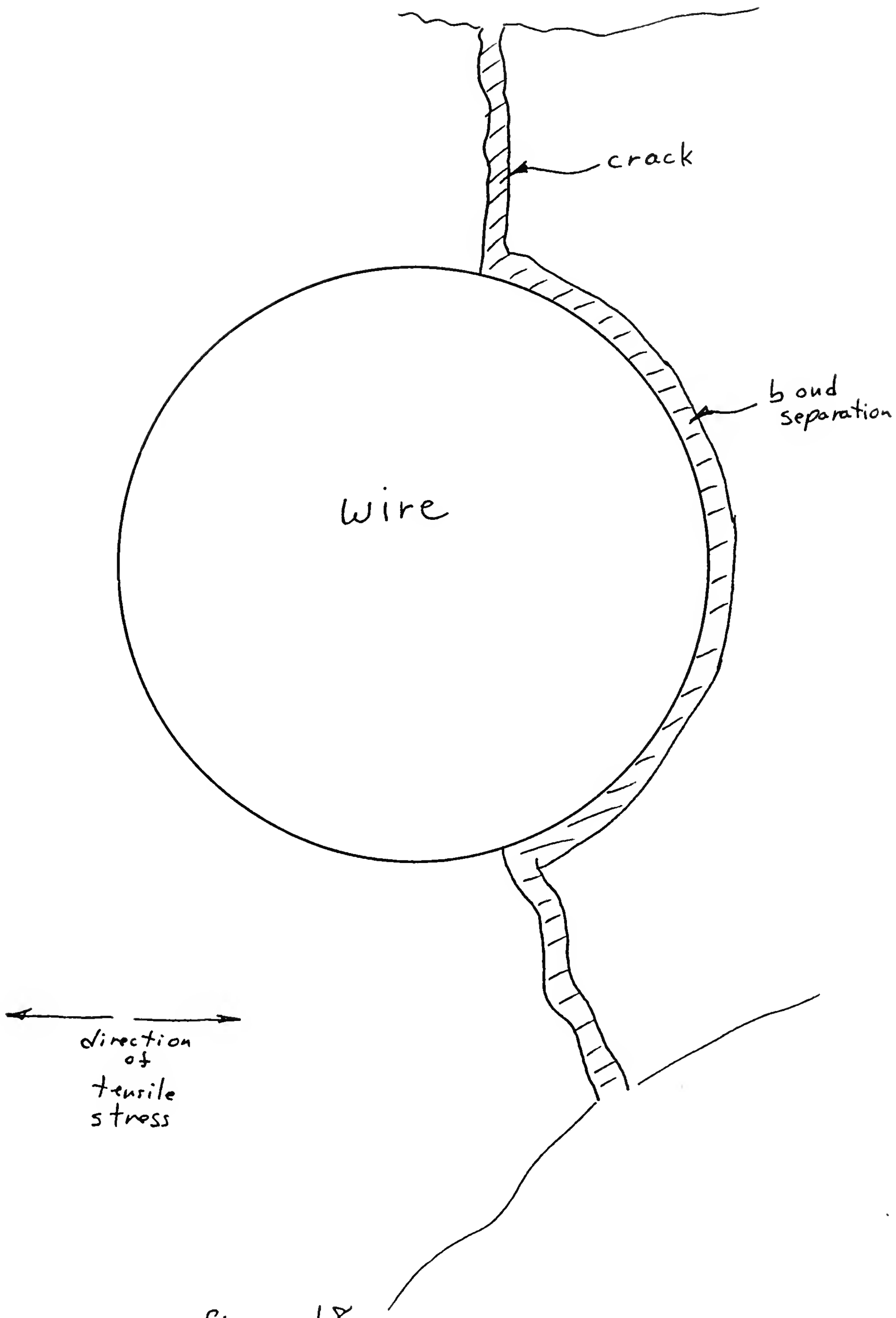
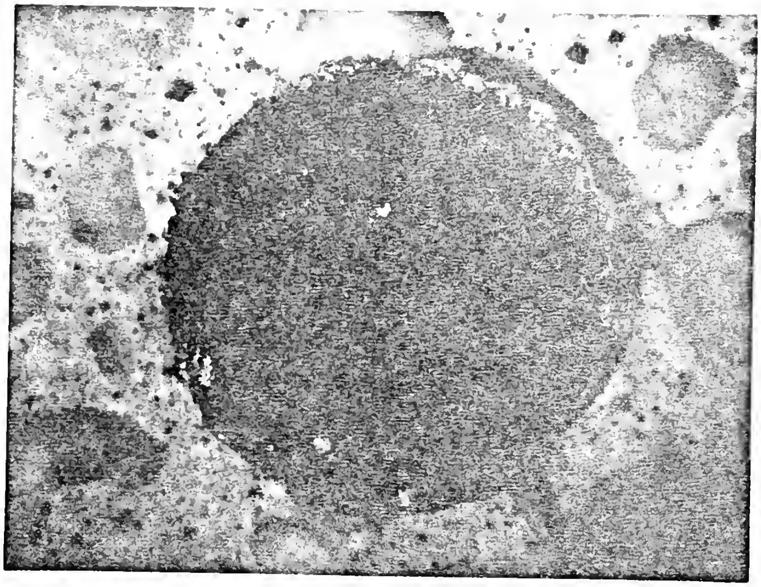
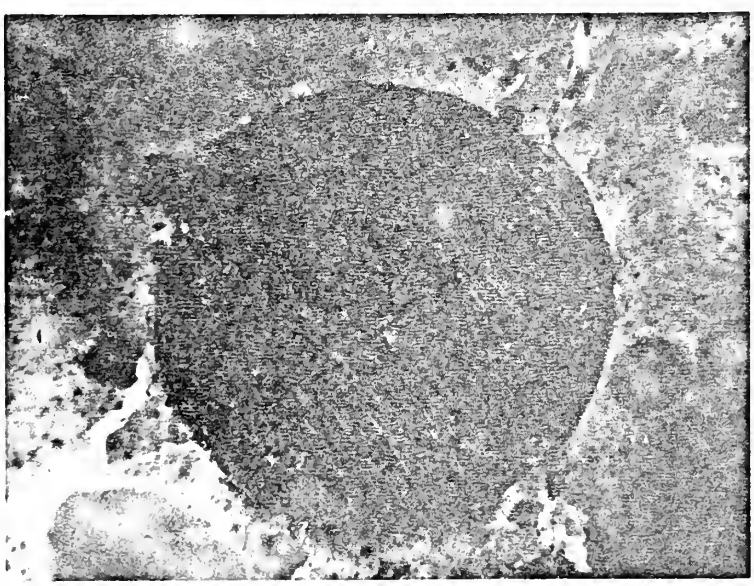



figure 18




a (X22)

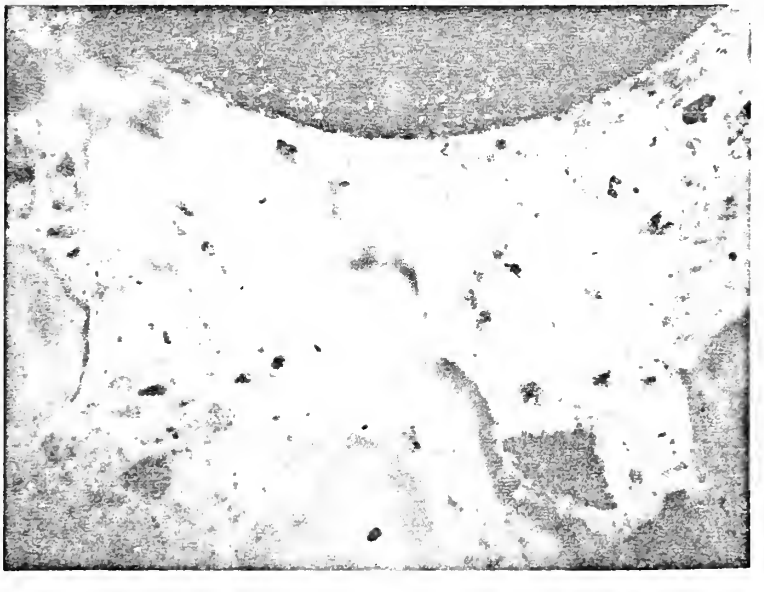



b (X22)

figure 19

tensile stress

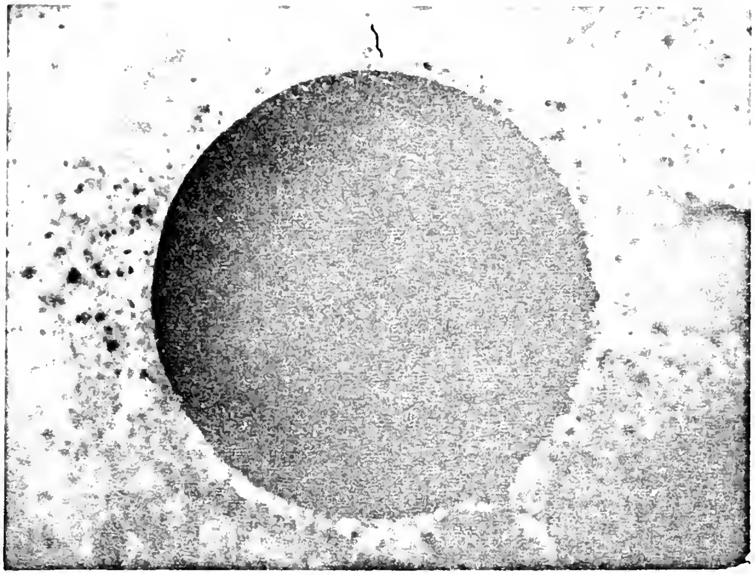
a (X 60)



tensile stress

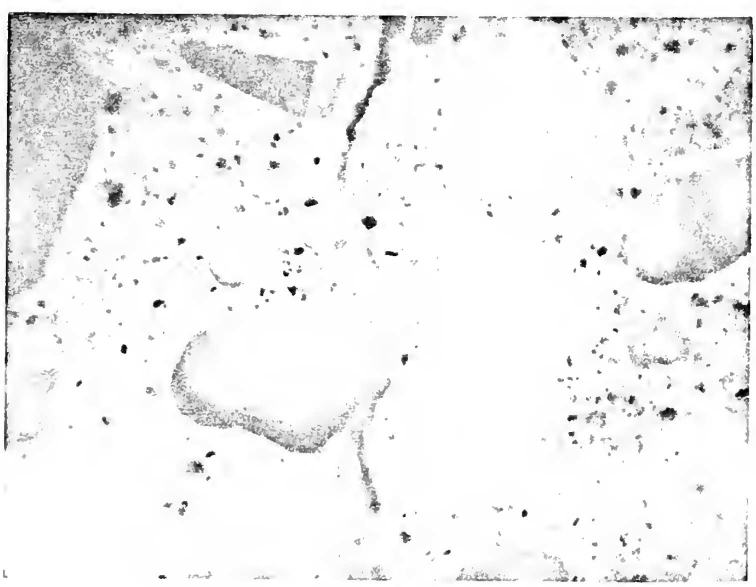
b (X60)

figure 20



tensile stress

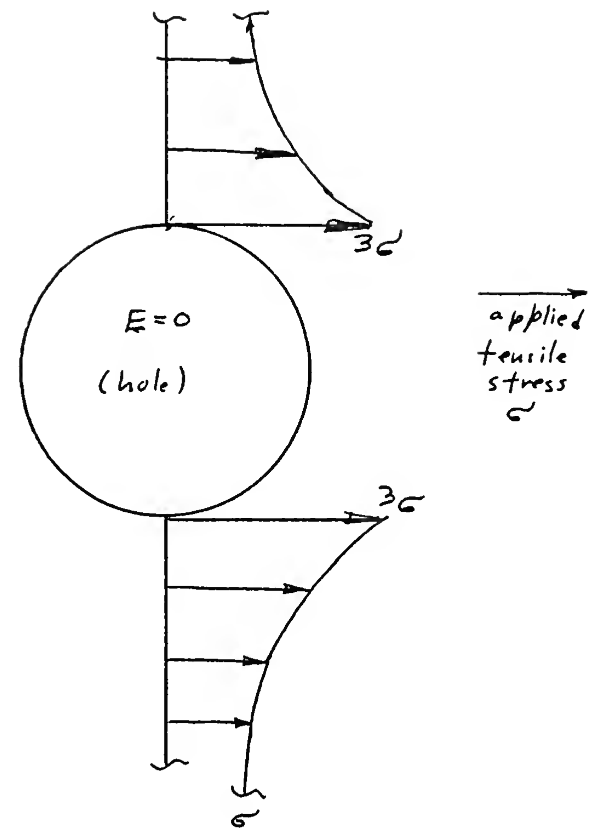
a (x22)

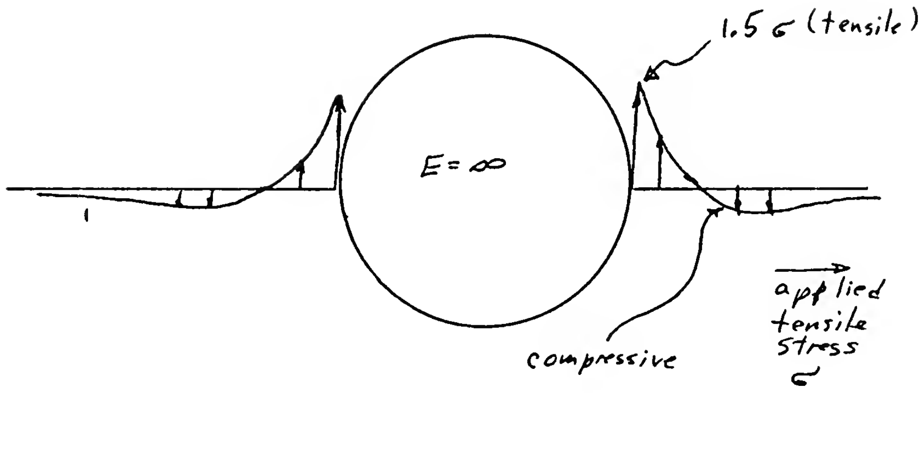


tensile stress

b (x44)

figure 21




Circumferential stress at maximum points for a hole and a rigid inclusion, after Goodier [30]

figure 22

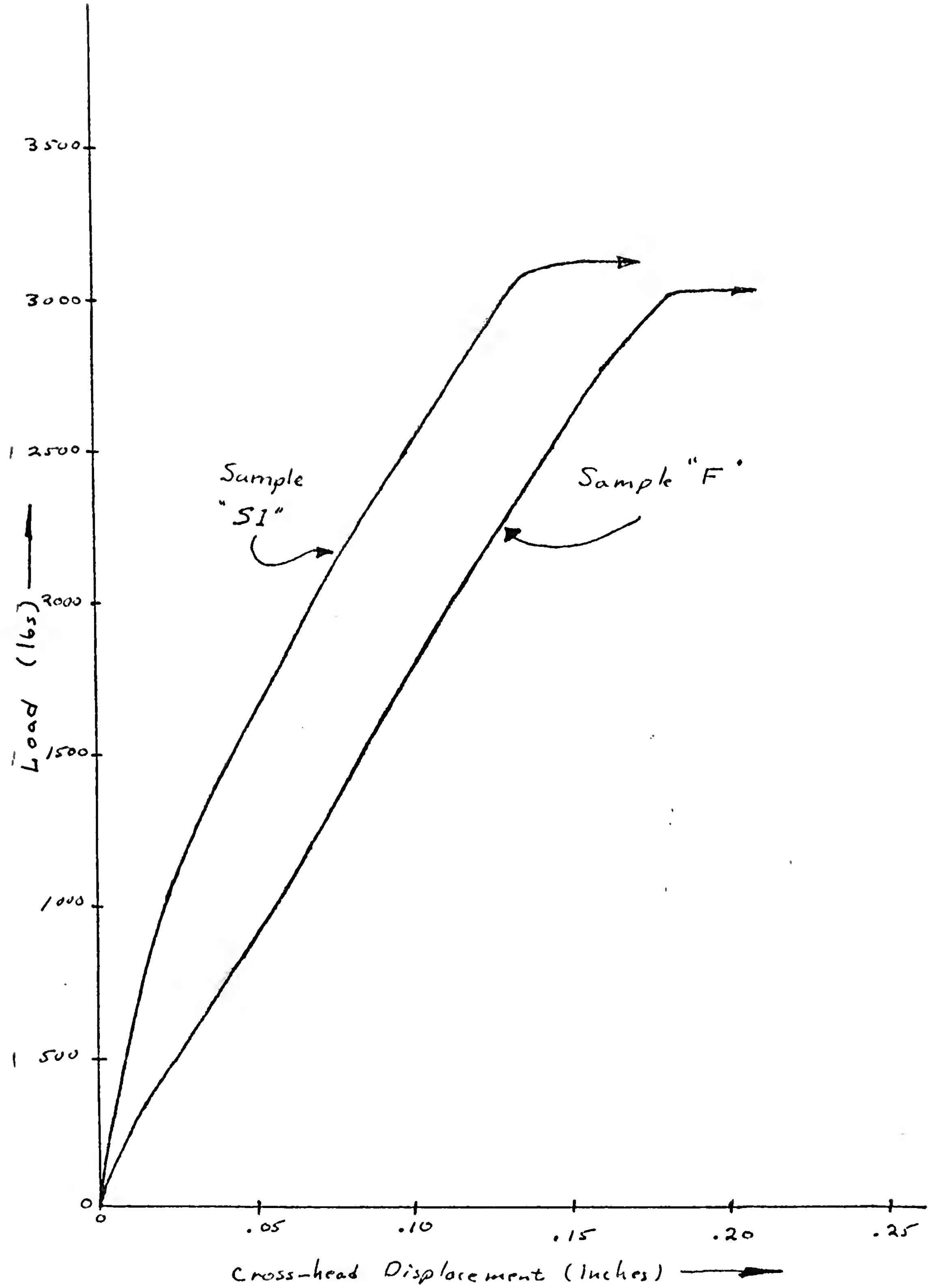
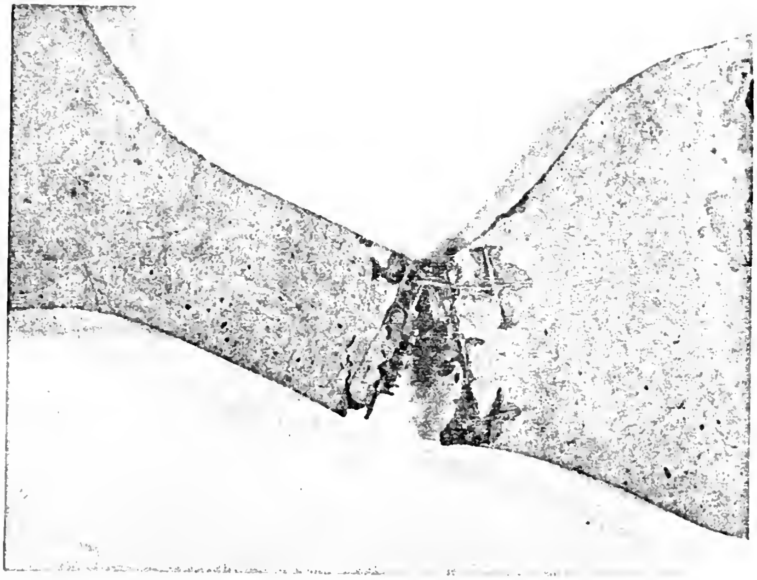

3500
3000
2500
2000
1500
1000
500
0
Load (lbs)
Sample "S1"
Sample "F"
0
.05
.10
.15
.20
.25
Cross-head Displacement (Inches)

figure 23



Untreated Sample


Treated Sample

figure 24